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Abstract Presented to the Graduate School of the University of Florida in Partial Fulfillment of the Requirements for the Degree of Master of Engineering

USING REINFORCED EARTH WALLS FOR BLAST MITIGATION

By

Ronald L. Pieri

3 December 1998

Chairman: Professor Frank C. Townsend
Major Department: Civil Engineering

The purpose of this report was to assess current blast mitigation methods. Several blast mitigation walls were constructed and tested to analyze their performance against force protection criteria. The three blast mitigation walls that were tested included a Hesko-Bastion Concertainer Revetment Module, a Maccaferri FlexMac Revetment Module and a Reinforced Earth Wall using Tensar Geotextiles.

The walls were sited and constructed in front of temporary living facilities. The temporary living facilities are the same size and type that are currently being used by soldiers and airmen deployed overseas. To test the capabilities of the blast mitigating walls, an explosive charge of 5,000 pounds was detonated. The walls and portions of the temporary living facilities were instrumented to determine the maximum reflected pressures that were generated.

There were two primary conclusions as a result of the test. First, current force protection methods must be implemented differently to minimize reflected pressures to 3 pounds per square inch. In addition, standoff distance is still the best method of minimizing blast effects. Second, in expedient construction environments, the prefabricated Hesko-Bastion and Maccaferri FlexMac revetment materials should be used. These materials make the construction of blast mitigation walls easier and faster than using reinforced earth materials.

BIOGRAPHICAL SKETCH

The author was raised in Sacramento, CA, where he excelled in soccer and rugby and dreamed of flying aircraft. After he obtained a private pilot's license and graduated from Del Campo High School, the next stop was the United States Air Force Academy in Colorado Springs, CO.

The Air Force Academy brought many changes to the author's life but none that could deter him from the original goal of flying aircraft. However, following back surgery in 1991, the pilot qualification rating was lost and another opportunity was born. After graduation in 1992, his first assignment was to Altus, OK. The author spent time in the Military Construction Program where he learned the role of the Air Force when working with the Corps of Engineers, the architect and engineering firms and the contractors.

Then, in 1994, the author had the opportunity to join the 823rd RED HORSE Squadron (RHS) located at Hurlburt Field, FL. While with the 823rd RHS, he performed countless construction evaluations, deployed to Baghdad, Iraq, in support of the UNSCOM mission, became qualified to handle and detonate explosives and mobilized with combat engineers to over ten countries within Europe and the Middle East where he led construction projects.

Finally, from 1997 until the present, he has been busy learning the intricacies associated with geotechnical engineering and pavement evaluation. When he leaves for Korea in 1999, the author will have plenty of time to apply both disciplines in a foreign environment.

USING REINFORCED EARTH WALLS FOR BLAST MITIGATION

By

RONALD L. PIERI

A REPORT PRESENTED TO THE GRADUATE SCHOOL
OF THE UNIVERSITY OF FLORIDA IN PARTIAL FULFILLMENT
OF THE REQUIREMENTS FOR THE DEGREE OF
MASTER OF ENGINEERING

UNIVERSITY OF FLORIDA

1998

"It is the soldier, not the reporter, Who has given us freedom of the press.

It is the soldier, not the poet, Who has given us freedom of speech.

It is the soldier, not the campus organizer, Who has given us the freedom to demonstrate.

It is the soldier, Who salutes the flag, Who serves beneath the flag,
And whose coffin is draped by the flag, Who allows the protester to burn the flag."

Father Denis Edward O'Brien, USMC

ACKNOWLEDGEMENTS

First of all, I'd like to thank my former commanders, who've put their faith in me and given me the opportunity to succeed and fail. Particularly Colonel Susanne Waylett, who reinforced the significance of "taking care of the troops", even when personal sacrifices were endured.

Second, my appreciation goes to the professors at the University of Florida. Their technical knowledge and ability to convey that knowledge to students is remarkable. Without the sacrifices that educators make, our future generations have little hope of contributing to the society that makes this country unique.

Third, to the men that provided an avenue for me to complete this report. Captain David Jurk with the 820th Security Forces Group allowed me to participate in the DIVINE BUFFALO V Test. This test gave me the opportunity to analyze current force protection measures and attempt to improve our blast mitigation techniques. Also, to William C. Dass, P.E., Randall W. Brown, Ph.D., P.E., and Kenneth J. Knox, Ph.D., P.E., with Applied Research Associates, Inc. These men gave me guidance throughout the DIVINE BUFFALO V Test.

Lastly, to my peers who have assisted me during the last 16 months. They have answered countless questions, explained engineering principles and problems and provided a source of motivation during the challenging times graduate school presents.

TABLE OF CONTENTS

LIST OF FIGURES

LIST OF PHOTOGRAPHS

LIST OF TABLES

SECTION I

INTRODUCTION

A. OBJECTIVE

The purpose of this project was to investigate the integrity of a reinforced earth wall subjected to blast loading. Historically, earth berms have been the favored structure for providing protection to soldiers against an attacking force. Typically the first structure constructed at an encampment is a berm system to repel vehicles, attacking soldiers or artillery directed at the defending forces. With the development of reinforced earth concepts and their applications in conventional civil engineering structures, an interesting question arose. The question was whether or not reinforced earth applications could be used for defending soldiers in foreign environments. To research this question a test program was developed to look at the following:

1. Establish the properties of reinforced earth when subjected to blast loads
2. Perform an actual test of a reinforced earth system subjected to blast effects
3. Compare the results of blast effects on a reinforced earth system with conventional blast mitigation structures

B. HISTORY

The end of the Cold War has greatly reduced the probability of the global nuclear war that was the principal threat since the 1950s. Following the Cold War, the threat has shifted to terrorists that are elusive and extremely dedicated. Terrorists' ability and dedication for strapping explosives to their backs and becoming martyrs for their cause has become increasingly alarming. Terrorist attacks have forced the Department of Defense (DOD) to create new systems to adequately protect military assets. The term

used for this process is force protection. Structures that are frequently hardened include aircraft shelters, command and control centers, ammunition storage areas and other functions that the base commander may specify. Dispersement and camouflaging are two other techniques used by military forces but these two methods will not protect assets if they are identified and attacked. Conventional weapons produce blast effects such as high-pressures, fragmentation of projectiles, cratering and spalling. These effects can completely negate a military force's ability to survive and counter-attack if defensive measures are not taken. Following a strike by an offensive force, it is in the best interest of the defensive force to reload, not rebuild. Therefore, current military doctrine requires constructing defensive structures to protect anything that is critical for counter-attack and survivability. Methods used for this purpose include constructing permanent structures such as reinforced concrete masses or burying critical assets well below ground level. The theory behind the second concept is that the soil separating the munitions entry point and the asset to be protected will dissipate the energy of the munition such that the asset is not destroyed. There have been extensive studies in this area and the concept will be discussed further in the literature review. However, this project will focus on reinforced earth walls for temporary defensive structures (one to five years).

When considering recent events that have caused leaders to reevaluate blast effects on structures, people frequently think of three events. The three events are the domestic terrorist bombing on the Oklahoma City Federal Building, the terrorist bombing on airmen living in the Khobar Towers in Saudi Arabia and the latest attacks on American Embassies in Kenya and Tanzania. The next couple of paragraphs will

summarize the Khobar Tower Bombing and the direction the military is moving to better protect its soldiers and airmen.

On June 25, 1996, a terrorist truck filled with explosives detonated outside the perimeter of Khobar Towers, Dhahran, Saudi Arabia. This facility was housing United States and Allied Forces supporting Operation Southern Watch, the mission for aircraft operations over Iraq. The size of the bomb has been estimated between 3,000 and 30,000 pounds of trinitrotoluene (TNT). However, the task force evaluating the bombing determined the explosion was more likely on the magnitude of 3,000 - 8,000 pounds and are using 5,000 pounds as the model to protect future structures against.

Figure 1: Khobar Towers After Bombing

The end result of the bombing for United States Soldiers and Airmen was 19 killed and approximately 500 wounded. Following the attack, the Secretary of Defense directed a

review of the facts surrounding the attack and the defensive measures undertaken prior to the attack. The review evaluated several items pertinent to the security of the United States forces. This report is focusing on how to minimize the damage of successful attacks.

One of the initial findings the task force discovered was that there were no published Department of Defense physical security standards for force protection of fixed facilities. Instead, they had published a limited number of handbooks (DoD O-2000.12-H) providing some suggestions towards force protection. Since there were no direct guidelines stipulating mandatory actions, commanders were left to a subjective determination of the threat. It was also determined that many commanders were unaware that the Department of Defense had provided handbooks for use in antiterrorism planning. The handbook was found in few locations where United States Forces were positioned. Although standards do not necessarily guarantee safety, they can establish a baseline for commanders to assess the threat and plan for base improvements.

The handbooks provided by the Department of Defense provided guidance on physical security for United States occupied facilities. It does not consider the structural characteristics of the buildings that should be protected. It does not define standards for design, materials, or construction of new buildings or modifications to existing buildings. Expedient and long term construction for enhancing force protection were oftentimes left up to the experience of the civil engineer and the money allocated. However, many commanders did not have an appreciation for the force protection measures required for adequate protection against terrorist attacks. In addition, the handbook gives some guidance for standoff distances for new construction but nothing for existing facilities.

Many of the commanders interviewed believed that at least 100 feet of standoff protection was required. Using 100 feet of standoff and a 5,000-pound detonation, structures would be subjected to reflected pressures in excess of 70 psi.

Following the review of the protective measures at the site of the blast and the guidance available to the commander, the following recommendations were made:

1. Establish prescriptive DOD physical security standards
2. Designate a single agency within DOD to develop, issue and inspect compliance with force protection physical security standards.
3. Provide this DOD agency with sufficient resources to assist field commanders on a worldwide basis with force protection matters. Consider designating an existing organization, such as a national laboratory, Defense Special Weapons Agency, or the Corps of Engineers, to provide this expertise.
4. Provide funds and authority to this agency to manage Research, Development, Test and Evaluation efforts to enhance force protection and physical security measures. (Record, 1996)

In the wake of these recommendations, my objective is to work in tandem with the DOD agency to develop a more protective wall system through reinforcement. Currently there are many contractors developing and evaluating new products for blast mitigation. However, these products will come to the DOD at significant expenses. Although the new concepts may be extremely valuable to expediently deployed forces, another solution may be incorporating existing civil engineering practices with current methods of constructing reinforced earth walls. The end result should be stronger walls, higher slope angles on the rear and end faces of the walls, less soil required for construction and a smaller footprint creating additional space for other purposes.

C. BACKGROUND

Increasing the survivability of structures by using soil as a protective shield has been used for a long time. The soil provides a barrier to reduce the pressure, heat and possible fragmentation resulting from explosions. The biggest disadvantage of using soil is the amount of space that is required. Since typical berm structures are constructed with slopes between 22° - 28°, a typical twelve-foot high berm will span thirty feet on the face of the berm. Considering the back face and the material between the faces, the berm's width can approach seventy feet. One of the principal advantages of reinforced earth concepts is that berm's slopes can be increased and the width of the entire berm will be greatly reduced. Typical reinforced earth walls use steel strips or geogrid materials to strengthen the soil system. Traditionally, "reinforced soil" is soil that is strengthened by a material's ability to reduce tensile stresses. Also, the material functions with the soil through friction. As stated in Engineering Principles of Ground Modification,

"The primary purpose of reinforcing a soil mass is to improve its stability, increase its bearing capacity, and reduce settlements and lateral deformation... many of the materials used to improve engineering properties of soil, such as geotextiles, can fulfill multiple functions, e.g., provide structural strengthening, control groundwater flow or accelerate consolidation with their drainage capacity, prevent particle migration through filter action, and maintain separation of different soil layers during construction or under the influence of repeated external loading" (Hausmann, 1990).

From the passage above, the desired properties for this project are to increase the berm's stability and to reduce lateral deformation. Some of the potential advantages of using reinforced soil are as follows:

1. Reinforced berms can be constructed using the same equipment sets traditionally found in Rapid Runway Repair Kits or Pre-positioned Assets.
2. Reinforced earth walls are cheaper and faster to construct than concrete structures.
3. Reinforced earth walls are cheaper to construct than current blast mitigating structures.
4. Reinforced earth walls do not create additional fragmentation hazards.
5. Reinforced earth walls will deform like typical unreinforced berms but will show better survivability against repeated external loads.
6. Reinforced earth walls will use less soil than traditional earth berms.

D. SCOPE OF TESTING

The objective of this test is to conduct a full-scale explosive test to show the effects of reflected pressures or airblast on lightweight, deployable structures. Also, the test will investigate the feasibility of blast mitigation barriers in protecting such facilities. This test is being performed because many Air Force Personnel at deployed locations live and work in expeditionary structures, such as tents or shelters, or in temporary structures, such as trailers or modular buildings. Force protection standards for these types of facilities are in various stages of development by the DOD, Regional Commands and Air Force Major Commands. Such standards require minimum standoff distances from possible detonation of a terrorist improvised explosive device (IED) of an assumed maximum size. If the standoff criteria cannot be met and the facility cannot be relocated, steps must be taken to mitigate the blast effect or strengthen the facility.

Both the developers of these standards and the engineers in the field who have to implement the standards need more information on the response of expeditionary and

temporary structures to large conventional blasts, ensuing casualties and the ways to mitigate damage if the appropriate standoff distance cannot be attained.

The DIVINE BUFFALO V test, scheduled for September 23, 1998, is an opportunity for obtaining data on the response and protection of expeditionary and temporary structures from a large IED. The detonation will be 5,000 pounds of C-4, which has a net equivalent weight of 6,400 pounds of TNT. The test is being sponsored by the Technical Support Working Group (TSWG) to investigate blast mitigation, managed by the Defense Special Weapons Agency (DSWA) and conducted at White Sands Missile Range (WSMR). The Explosive Test on Lightweight, Deployable Structures is cosponsored by the Force Protection Battle Lab, the Air Force Research Laboratory and the U.S. Army Soldier Systems Command. The agencies involved and their responsibilities are as follows:

1. Force Protection Battle Lab: Sponsoring agency and overall project manager.
2. Air Force Research Lab, Air Base Technology Branch: Performing agency and test sponsor, providing manpower and responsible for test coordination, test article procurement, instrumentation planning, data reduction and analysis and test documentation.
3. U.S. Army Soldier Systems Command: Test sponsor, providing test articles from their Force Provider assets and financial support.
4. Technical Support Working Group: Sponsor for DIVINE BUFFALO test series.
5. Defense Special Weapons Agency: Overall program management for DIVINE BUFFALO test series.
6. Field Command, Defense Special Weapons Agency: Test project manager for DIVINE BUFFALO test series at White Sands Missile Range, NM.
7. 49th Materiel Maintenance Group: Technical assistance and test article acquisition and construction.

8. PMR Construction Services: On-site contractor supporting construction

As stated before, this test will show the performance of several test articles. The structures that will be analyzed include three walls, four tents and two general-purpose shelters. Several of the structures will be situated behind the blast mitigation walls. The wall heights are two feet greater than the structure heights and the wall lengths are four feet longer than the structure lengths. The purpose of the blast mitigation walls is to limit the pressure felt by the facilities to less than 3 psi. The following walls will be compared:

1. Hesco-Bastion Concertainer Revetment Modules
2. Maccaferri FlexMac Revetment Modules
3. Reinforced-earth wall using TENSAR geogrid reinforcement

There is an extensive instrumentation plan to measure and record the effects of the blast. The plan calls for twenty airblast pressure gauges and three high-speed cameras. The cameras and data acquisition system will be tied into the White Sands Missile Range event timing system. Video footage and 35 mm color photographs will be taken of test article construction, test configuration, exterior and interior of the structures, blast mitigation walls, the charge, instrumentation set-up and post-test configuration of the test articles. The following pages represent the overall site plan and instrumentation plan for DIVINE BUFFALO V. Applied Research Associates developed the test layout.

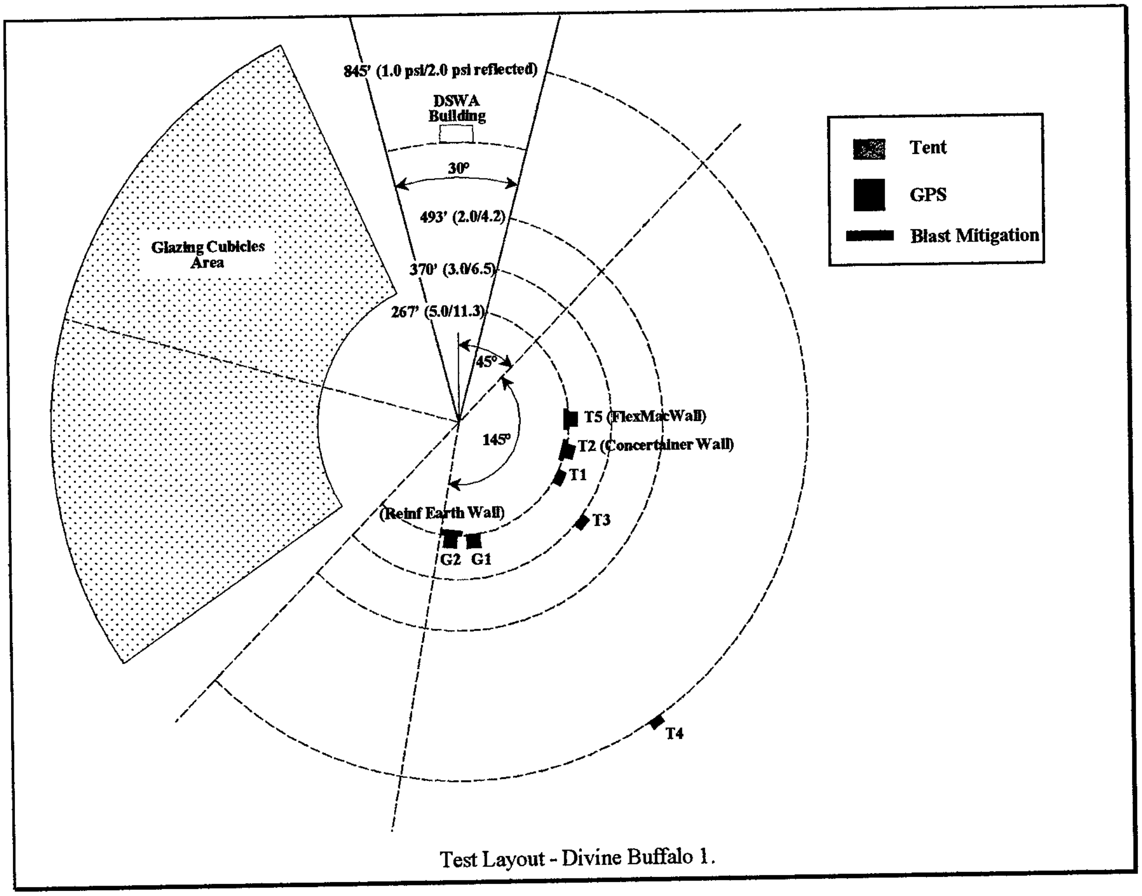


Figure 1.2: DIVINE BUFFALO V Test Layout (Knox, 1998)

The primary focus of this report is to evaluate the performance of the reinforced earth wall. The reinforced earth wall is situated in front of test article G2.* The figure also shows test article G1 directly next to G2. Test article G1 is a facility unprotected from the blast wave and will act as one of the controls in this test. The other two walls are formed from modules and will be constructed much like a gabion or geocell wall. They are situated such that they protect a facility placed directly behind the wall. Like the reinforced earth wall, there is an unprotected facility on the side of the module walls acting as a control. At the conclusion of the test, the criteria that will be compared includes material costs, equipment required for construction, time required for

construction, deformation of the mitigating barriers and the recorded pressure on the facilities located behind the test articles.

E. INSTRUMENTATION

The instrumentation plan is supported with pressure gauges and high-speed cameras. The goal for the pressure gauges is to record the reflected pressure following the detonation of the C-4. Of primary interest is to record the pressure values on the faces of the barriers, the facilities that are situated behind the barriers and the facilities that are unprotected. The distance between the explosion and the barriers is 267 feet. Using cube root scaling, which will be discussed in the literature review, the reflected pressure can be estimated using the following model:

$$R/W^{1/3} = R_2 / W_2^{1/3} \qquad (1)$$

$R/W^{1/3}$ = scaled distance and represented by λ

W_2 = actual explosion net equivalent weight

R_2 = actual distance of the explosion to the point of interest

or

$$\lambda = R_2 / W_2^{1/3} = 267' / 6400^{1/3} = 14.38 \qquad (2)$$

Using Figure 3-7 from TM 5-855-1, this gives a reflected pressure roughly equal to 12 psi. The pressure gauges located on the test articles will allow a comparison between the estimated reflected pressure and the actual reflected pressure acting on the walls and facilities. There are twenty pressure gauges, sixteen that will be mounted on the blast mitigating walls and the shelters, and four that will be for free-field readings.

From Ross, 1998, the pressure gauges require a (–3 dB) signal Bandwidth of 50 kHz. The full bandwidth of the signal conditioning amplifiers is 100 kHz. But, since the bandwidth requirement of the pressure gauges is 50 kHz, a 50 kHz lowpass filter is required on the front end of the signal conditioning amplifiers. The recording rate of the pressure sensors is 256 Ksamples/sec (4.0 x 10^{-6} sec/pt) giving a total duration of 0.4587 seconds.

The recording station is a fixed facility that will be protected from the blast. The station accommodates 100 data channels plus 20 dual channels. Each data channel consists of one signal conditioning amplifier and one transient data recorder. The data acquisition system is connected to a computer system that will record the entire event. Each transient data recorder has a memory of 128K words. The total memory is divided into 16 segments, each segment contains 8,192 words. Segment 0 will be dedicated for the calibration cycle, segment 1 is allocated for the pre-data accumulation and segments 2-15 are responsible for collecting the actual data.

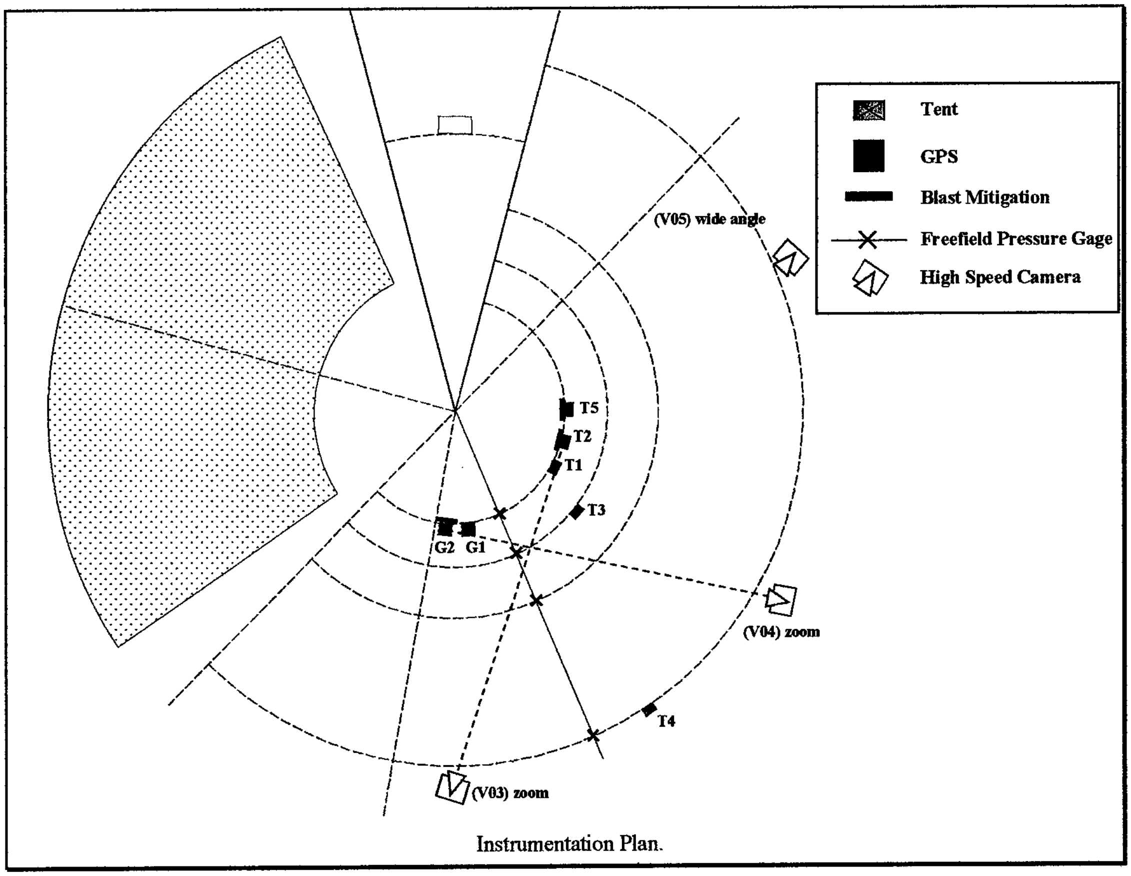


Figure 1.3: DIVINE BUFFALO Instrumentation Plan (Knox, 1998)

The firing sequence is crucial for the data acquisition system. The initial time (T_0) will be initiated from a 28-volt direct current pulse that will activate the X-UNIT. Two independent signals are output from the X-UNIT to the recording bunker to indicate T_0 to the transient data recorders. Concurrently, a high voltage will be generated by the X-UNIT to detonate the explosive charge. The components of the timing system will include the sequencer, a trigger generator, an interface control unit and a monitor abort system. The firing system includes the connection of the X-UNIT to the actual charges and all of the support wiring and components related to the detonation. From the X-UNIT to the trigger generator, there are two trigger cables receiving the inductive coil

pickups. It is crucial that each of the two cables come from the two separate trigger output sources from the X-UNIT into two separate inputs of the trigger generator. This ensures that at T_0, there will be a trigger signal to start the transient data recorders. Before each event, it is also necessary to trigger the X-UNIT to make sure that a trigger signal is present to start the transient data recorders. This will be accomplished by testing the first trigger line, and then testing the second trigger line, to make sure that both lines are working independently. After each trigger line is checked individually, both trigger lines will be tested simultaneously. The monitor abort system will stop the sequencer (and the test) if any of the following occur:

1. A bank of digitizers indicates that at least one digitizer has failed
2. The interface control unit is not ready to initiate the control commands
3. The data acquisition computer is not ready to accept data
4. There is a spontaneous trigger

If a specific gauge fails immediately prior to T_O, it will not be identified by the monitor abort system.

SECTION II

LITERATURE REVIEW

A. OBJECTIVE AND STRUCTURE

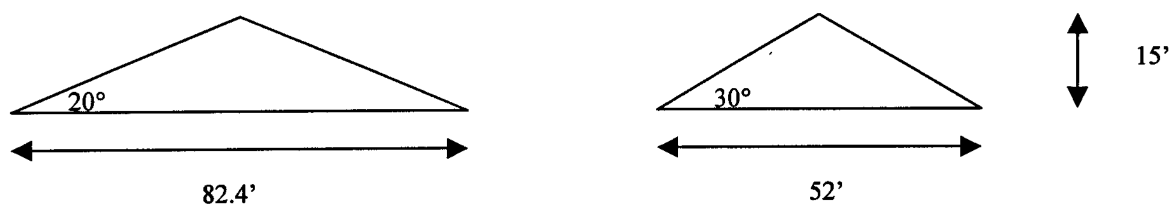
This section discusses the findings of the literature review. There are several areas associated with reinforcing cohesionless soils that are subjected to blast loading. The areas to be analyzed are as follows:

1. Using earthen berms to resist blast loading.
2. Using reinforced earth walls to resist blast loading.
3. Airblast Effects
    a. Cube-Root Scaling
    b. Blast Wave Phenomena

B. USING EARTHEN BERMS TO RESIST BLAST LOADING

The idea that reinforced soil may resist blast loading stems from the historical performance of unreinforced soil's ability to resist blast effects. Unreinforced earth berms are currently used in expedient construction measures to protect critical assets. More often than not, the soil is placed against existing walls to minimize the effects of near-miss attacks. The soil's mass prevents damage from the reflected pressure generated by the explosion and stops fragmentation from penetrating the structure that is being protected. The benefits of using soil are numerous. The first is that in most places, soil is readily available and can be collected from the actual site or it can be purchased from local contractors at a relatively inexpensive price. The second is that soil is easily molded to any structure or site condition and can protect a variety of assets. The third is that it can be placed very quickly in hostile environments. The berms can be constructed

using heavy equipment such as backhoes or loaders, or they can be formed less quickly using men, shovels and sandbags. However, the biggest disadvantage of using soil for freestanding berms is the footprint required for construction (Sues et al., 1989). Because the faces are constructed between 20° - 30°, a 15-foot high wall will extend a minimum of 82.4'- 52' respectively.



For the space required to construct these berms, they may become unacceptable for certain applications. This is because a blast wave hitting one side of the berm may curl down the other side and damage the asset the berm was supposed to protect.

Many studies have been performed to analyze the benefits of using protective earth berms in comparison with other expedient hardening methods (Sues et al., 1989). Coltharp et al., 1985, reported the comparative performance of earth berms, spall plates and increased wall thickness as another way of reducing the effects of spall induced by blast loading on reinforced concrete boxes that were 5.4 feet high with wall thickness varying from 12.8 inches to 21.7 inches. Measured peak pressure against the wall averaged about 9,400 psi in six tests without berms and approximately 1,200 psi in two tests with berms. Maximum midspan deformations were also reduced by the presence of berms. Boxes with spall plates deformed approximately 1.2 inches, while boxes with thickened walls deformed 0.6 inch. Boxes shielded with earth berms deflected only 0.2 inch. In addition, peak accelerations were reduced by at least 50 percent and in some

cases as high as 90 percent by earth berms. Coltharp et al., 1985, concluded that, "berming permits the use of lower steel ratios, produces a more flexural-type response, and is the most cost effective solution"

In 1989, Hyde discussed the effects of full-scale testing on a reinforced concrete structure above the ground. Peak pressure, acceleration and wall deformation were compared between sand grids, Bitburg revetments, precast concrete panels and a sand berm. All four methods were successful in eliminating spall effects and peak pressures and accelerations were reduced by more than an order of magnitude. The amount of deformation also decreased slightly. Although the sand berm did not show the greatest reduction in pressure, acceleration or deformation, it stayed mostly intact following the test. In comparison, neither the revetments nor the precast panels survived the blast sufficiently enough for any second-strike protection. Hyde concluded that a sand berm is the most cost-effective solution.

Sues et al., 1989, analyzed the data on a broader scope of hardening methods than the previous two studies. In addition to face panels and modular structures, the studies analyzed four other structures that represented earth structures. This study looked at earth berms, sand grids, sandbagging and bin revetments. The study concluded that the most effective blast mitigating structure was the earth berm. While the other structures required less space, they did not give any second strike protection. This is because the structures rely on the stability that comes from the materials they are formed from. Once the plastic forms, sandbags and bins were deformed, the structures lost their integrity.

C. AIRBLAST EFFECTS

An understanding of the blast effects created by an explosion is critical when working with explosives testing. This section is going to define blast effects, net equivalent weight (NEW) and cube-root scaling. Also, a table giving the equivalent weights of common military explosives will be presented. Finally, there will be a discussion on blast wave phenomena and the parameters of a surface-burst.

<u>Definitions and Properties</u>:

The US Army Technical Manual, 1986, has the following definition for blast effects:

> "The blast effects of an explosion are in the form of a shock wave composed of a high-pressure shock front which expands outward from the center of the detonation, with the pressure intensity decaying with distance. As the wave front impinges on a protective structure, a portion of the structure or the structure as a whole will be engulfed by the shock pressures. The magnitude and distribution of the blast loads on the structure are a function of the type of explosive material, weight of explosive, the location of the explosion relative to the protective structure, and the interaction of shock front with the ground or the protective structure itself."

There are many different kinds of explosives. They vary in color, smell, detonation rate and generated heat. For example, the detonation rate of ammonium nitrate fuel oil (ANFO) and commercial dynamite is extremely slow when compared to C-4 or Pentaerythritoltetranitrate (PETN). That is why ANFO or dynamite is traditionally used in the mining industry. The blast effect generated by the detonation gives a "pushing effect" because of its comparatively slow detonation rate. In contrast, demolition experts within the military and civilian industry prefer C-4 because of its rapid detonation rate and "cutting effect". It is the preferred explosive for cutting steel or

other reinforcing materials. Because of the great variety of explosives, TNT is the standard reference. The correlation in a US ARMY Technical Manual, Fundamentals of Protective Design for Conventional Weapons, 1986, states:

> "The free-air equivalent weight of a particular explosive is the weight of the standard explosive TNT required to produce a selected shock wave parameter of magnitude equal to that produced by a unit weight of the explosive in question."

The Army's Technical Manual also gives a table showing the averaged free-air equivalent weights based on blast pressure and impulse.

Figure 2.1: Equivalent Weights of Explosives

Explosive	Equivalent Weight Pressure	Equivalent Weight Impulse	Pressure Range (psi)
ANFO	.82	-	1-100
Composition A-3	1.09	1.07	5-50
Composition B	1.11	0.98	5-50
Composition C-4	1.37	1.19	10-100
Cyclotol	1.14	1.09	5-50
HBX-1	1.17	1.16	5-20
HBX-3	1.14	0.97	5-20
H-6	1.38	1.10	5-100
Minol II	1.20	1.11	3-20
Octol	1.06	-	-
PETN	1.27	-	5-100
Pentolite	1.42	1	5-100
Tetryl	1.07	-	3-20
Tetrytol	1.06	-	-
TNETB	1.36	1.1	5-100
TNT	1	1	
Tritonal	1.07	0.96	5-100

This means that to achieve the same shock wave, a terrorist could use 73 lbs. of C-4 instead of 100 lbs. of TNT (100 / 1.37 = 72.99).

Cube-root or Hopkison scaling is used to convert the airblast characteristics of an explosion from one amount of energy to a different amount of energy. "According to cube-root scaling, a given pressure will occur at a given distance from an explosion that is proportional to the cube root of the energy yield. This has been proven true experimentally for explosive weights ranging from a few ounces to hundreds of tons" (TM 5-855-1, 1986). The model can be extremely useful for research applications because engineers are able to test large-scale detonations with small amounts of explosives. For example, rather than testing a 10,000-pound (NEW) shot at 600 feet, a test can be performed using 500-pounds (NEW) at a distance of 220 feet. The scaling relationships are applicable when the tests are done under identical ambient conditions, the same explosive configuration and identical charge-to-surface layout. However, if the above criteria are similar rather than identical, reasonable values are still obtained.

Blast Wave Phenomena:

An explosion in a gaseous medium creates an instantaneous and dramatic pressure increase in that particular medium. The change in pressure is called a blast wave, and immediately raises the ambient pressure to a peak incident pressure. When an explosive ignites, the initial response is compression of the explosive to a high density, creating a shock wave. The next step in the reaction is that the explosive is transformed into a high-temperature gaseous substance. This reaction can be compared to the pistons in an engine because a continuous shock wave is propagated at a constant velocity.

Mathematically, there is an equation that represents the conservation of mass from which the detonation velocity can be determined.

Mass:

$$\frac{D}{v_1} = \frac{D - W_2}{v_2} \qquad (3)$$

Where:			
	D	=	velocity of detonation
	W_2	=	velocity of material behind the wave, relative to front
	v	=	specific volume
	1	=	explosive's initial state
	2	=	explosive's final state

Equation (3) can be solved to obtain the detonation velocity:

$$D = v_1 \{ [(p_2 - p_1) / (v_1 - v_2)]^{1/2} \} \qquad (4)$$

Using this equation, calculated and measured velocities for common explosives are found in the following table (Fordham, 1980):

Figure 2.2: Detonation Velocities of Explosives

Compound	Velocity of Detonation (ft/sec)	
	Calculated	Maximum Observed
Nitroglycerine	26437	26240
PETN	26732	24928
Tetryl	24764	23944
TNT	21254	21976
Nitroguanidine	13251	12628
ANFO	11349	-

While equation (4) allows the maximum velocity expected of a particular explosive to be calculated, it does not predict the response of the detonation with respect to pressure

versus time. After the incident pressure is generated (milliseconds after the detonation), the pressure wave travels radially from the detonation point. The incident pressure travels on the surface parallel to the direction of the blast wave.

> "This pressure increase or shock travels radially from the burst point with a diminishing velocity, U, which is always in excess of the sonic velocity of the medium. Gas molecules, making up the front, move at lower velocities, u. This latter particle velocity is associated with a dynamic pressure or the pressure formed by the winds produced by the shock fronts. As the shock front expands into increasingly larger volumes of the medium, the peak incident pressure at the front decreases and the duration of the pressure increases" (TM 5-855-1, 1986).

The next diagram represents the pressure disturbance at any point away from the burst.

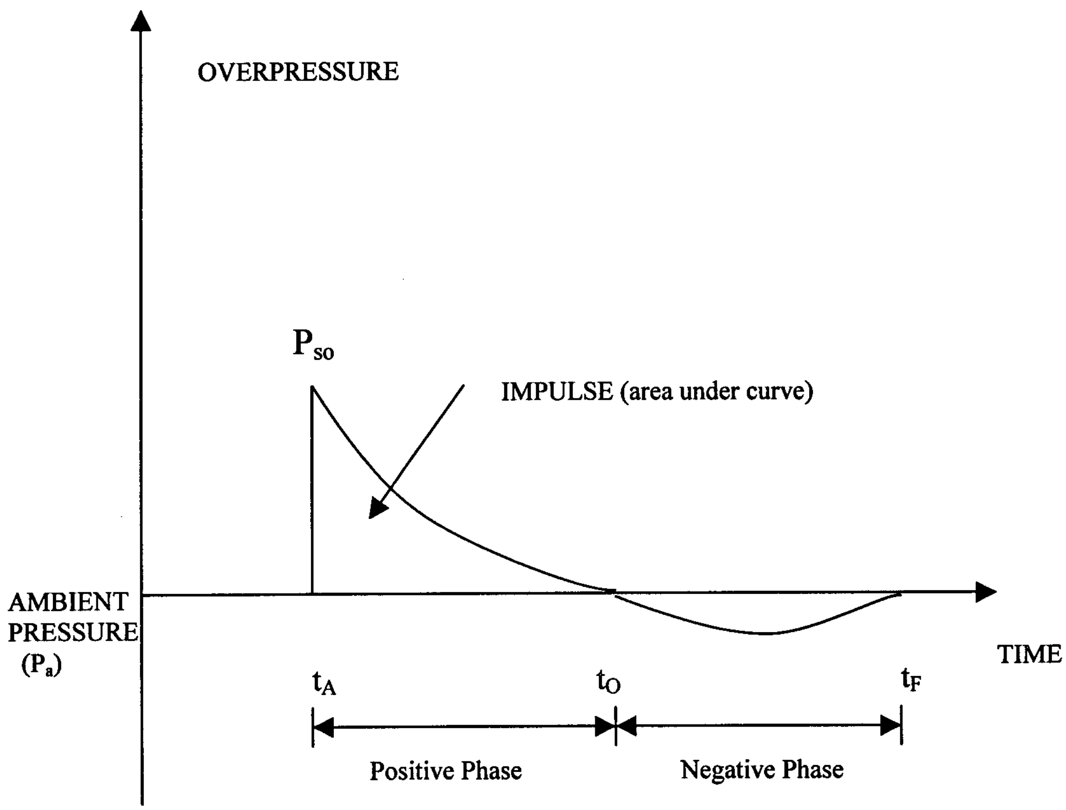


Figure 2.1: Reflected Pressure-Time History

Where: t_A time when shock front arrives

t_O time when shock front has peaked and decayed to ambient value

The negative phase of the diagram represents a pressure lower than the ambient pressure and usually lasts longer than the positive phase.

If the shock wave approaches a fixed surface situated at an angle to the direction of the propagated wave, a reflective pressure develops on the surface that is in excess of the incident pressure. The reflected pressure is dependent upon the magnitude of the incident pressure and the angle between the fixed surface and the wave propagation. The reflected pressure can be ten times greater than the incident pressure. The next figure comes from TM 5-855-1, 1986, and represents a hemispherical shock wave.

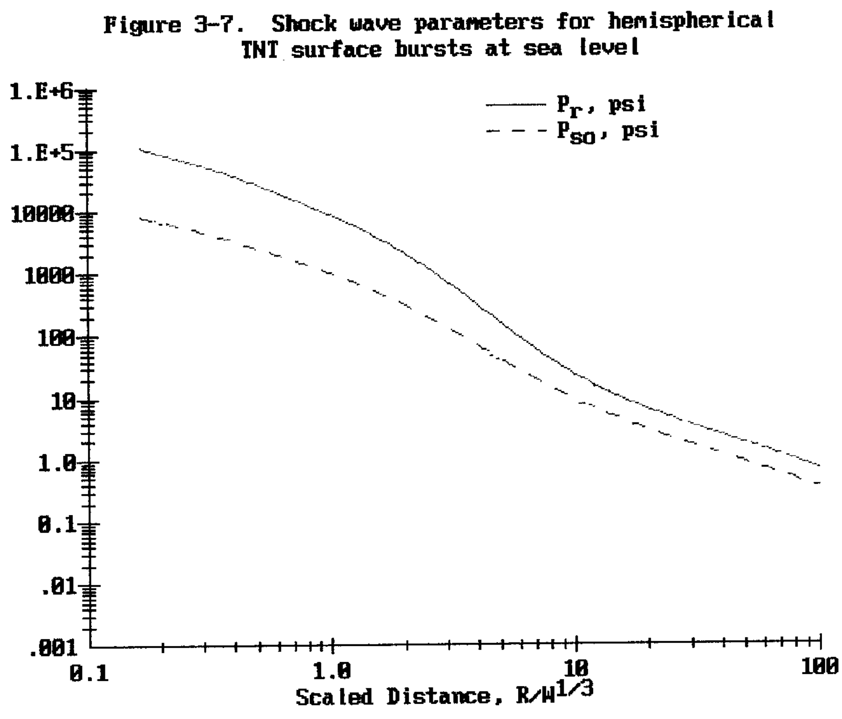


Figure 2.2: Shock Wave Parameters for Hemispherical Surface Burst. The x-axis represents the scaled distance and the y-axis represents pressure (psi).

The reflected pressure (P_r) reaches a maximum when the incident pressure (P_{so}) approaches a structure that is perpendicular to the blast wave's direction of travel. Conversely, the minimum reflected pressure is equivalent to the incident pressure and is reached when the reflecting surface is parallel to the blast wave's direction of travel.

Surface Burst:

A surface burst is a charge that is detonated on or very near the surface. "The initial wave of the explosion is reflected and reinforced by the ground surface to produce a reflected wave. Unlike an airburst, the reflected wave merges with the incident wave at the point of detonation to form a single wave similar in nature to the reflected wave of the air-burst but essentially hemispherical in shape" (TM 5-855-1, 1986). The next figure describes the effects of a surface burst upon a structure.

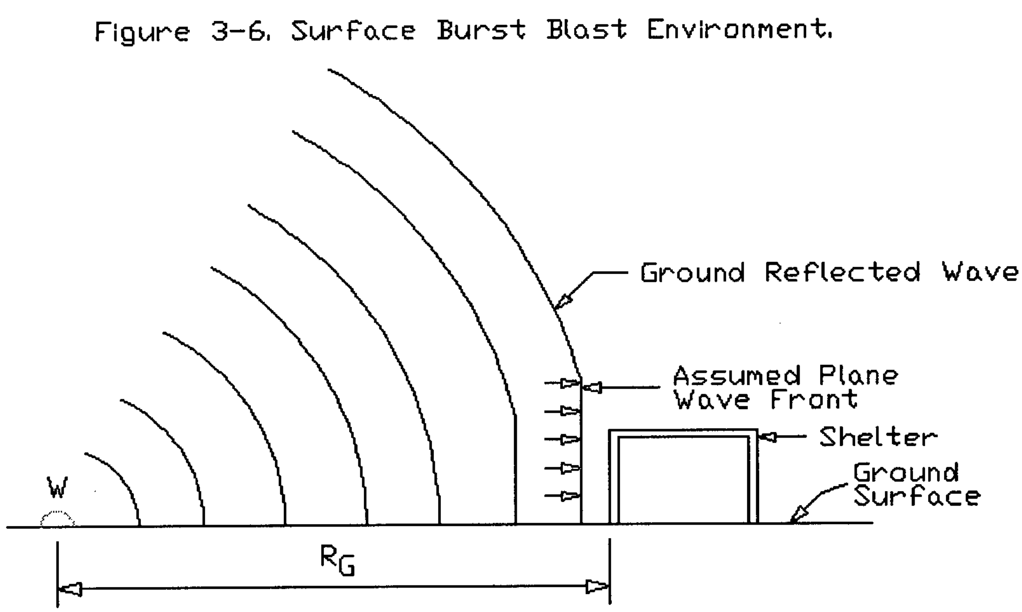


Figure 2.3: Surface Burst Blast Environment

The figure shows that through blast effects and the detonation near the proximity of the surface, the blast wave is reflected rather than incident. This is an important distinction because of the difference in pressure. Figure 2.2 shows that at a scaled distance of 1.0, the incident pressure (P_{so}) is around 1000 psi while the reflected pressure (P_r) is approximately 9500 psi.

Blast Wave Importance:

The importance of understanding blast wave phenomena is most important when lives and resources are threatened. The real challenge is applying blast wave phenomena when siting facilities in dangerous environments. After calculating the maximum pressure expected from an assumed net equivalent weight of explosives, it is important that engineers and technicians allocate adequate standoff distances to protect all assets. Problems arise when adequate standoff distances are not available. When this occurs, an understanding of blast mitigation is crucial so assets can be protected using barriers, walls or earth. The following table shows blast overpressure effects:

Table 2.3: Blast Overpressure Effects on Structures and Equipment

Target Element	Damage	Blast Overpressure (psi)
Glass Windows	Shattering, occasional frame failure	0.5 - 1.0
Large and Small	Severe frame failure	1.5 - 3.0
	Roof rafters cracked	0.5 - 1.3
Wood Frame	Studs and sheathing cracked	1.0 - 3.0
Structures	Collapse	Over 5.0
Metal Buildings	Corrugated aluminum/steel paneling moderately buckled/joints separated	0.5 - 1.0
(Butler Type)	Severe buckling/some panels torn off	1.0 - 2.0
	Complete destruction of siding interior destroyed	Over 3.0
Concrete Block or Brick Wall, 8-12"	Severe damage, shattering	1.0 - 2.0
(unreinforced)	Collapse	7.0 - 8.0
	Moderate cracking	3.0 - 4.0
Reinforced	Severe spalling & wall displacement	6.0 - 8.0
Concrete Walls	Concrete shatters, bare steel remains	10.0 - 14.0
	Complete destruction	14.0 - 20.0
Vehicles/Trailers	Complete destruction	10.0 - 14.0
	Moderate damage	6.0 - 8.0
Heavy Machinery	Complete displacement	8.0 - 10.0
(generator/compressor)	Destruction	14.0 - 20.0
Steel Towers	Blown Down	Over 30
Personnel	Temporary ear damage	0.2

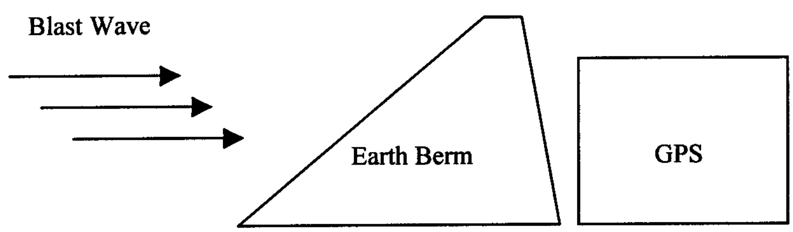
It is interesting to note that expediently constructed structures will all collapse at pressures over 5.0 psi. Also, pressures greater than 1.0 psi will shatter windows in any structure. This is an important consideration for force protection because most of the injuries resulting from the Khobar Towers Bombing were caused by glass. Also, while the exact damage to cause severe injury or death is not available, ear damage at 0.2 psi will greatly reduce any person's ability to perform a job.

SECTION III

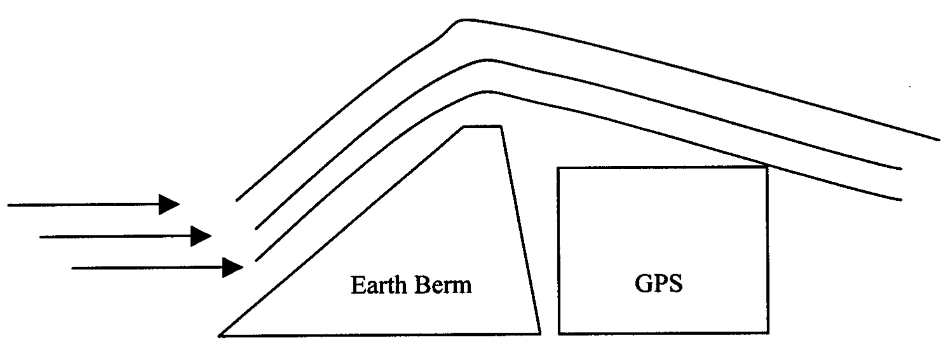
REINFORCED EARTH WALL DESIGN

A. OBJECTIVE

The purpose of this section is to describe the parameters used for designing a reinforced earth wall system. The major advantage of using reinforcement is to limit the footprint of the berm by constructing a near 90° face. The steep angle allows the facility to be protected from the blast wave effects. The following diagram represents the wall system the author attempted to construct for protecting a general-purpose shelter (GPS):



The near-vertical face was necessary to protect the GPS. Since blast waves will eventually curl over the berm and realign their direction with the surface of the earth, a protective "bubble" must be formed. The above system is intended to redirect the blast wave in the following manner:



B. DESIGN

The design was accomplished using Tensar's design method titled, "Slope Reinforcement with TENSAR Geogrids Design and Construction Guideline" (TENSAR, 1988). This design method enables soil slopes to be constructed using any slope angle with soil located at the site. The design method attributes its ability to construct slopes to the properties of the geogrid reinforcement. Some of the properties discussed include long-term allowable design strength, high tensile modulus at low strain levels, high interlocking capability and a service life in excess of 100 years. These properties have a crucial role in determining the stability, economy and performance of reinforced soil systems.

The assumptions made when using this design method are:

" 1. The soil is reinforced with horizontal layers of TENSAR Geogrids.
2. A ϕ' only ($c' = 0$) analysis is appropriate. This would apply to all gravelly and sandy soils and for long-term stability of silty and clayey soils.
3. The soil has uniform strength properties throughout the entire slope.
4. The slope face is planar and the top of the slope is horizontal.
5. Positive drainage is provided to assure that pore water pressure in the slope is zero.
6. No seismic forces are acting.
7. The slope foundation is competent.
8. Surcharge loads, if any, act uniformly on the top of the slope" (TENSAR,

1988). Other important values associated with geotechnical design are soil and geometry parameters. The soil parameters that must be estimated or determined are moist unit weight (γ) and the soil friction angle (ϕ'). Cohesion in the soil is assumed to be 0 psf. The global geometry to be considered include the slope's height (H), the slope's angle (β) and whether or not a surcharge (q) is present. Internal geometry elements that must be

designed include the vertical spacing between geogrid layers (s_{min}), the lift thickness (v) and the length of the geogrid layers.

The process of determining the required number of TENSAR Geogrid Layers is obtained from the TENSAR guidelines:

"1. Calculate the modified slope height, H', in order to account for surcharge.

$$H' = H + q/\gamma$$

This is valid only if H' is less than 1.2(H).

2. Calculate the factored soil friction angle, ϕ'_f (degrees). This is how the desired overall slope factor of safety, FS, is incorporated into the design. Slope stability factors of safety of 1.3 to 1.5 are typically accepted factors of safety for standard geotechnical engineering practice.

$$\phi'_f = \tan^{-1} (\tan \phi'/FS)$$

3. Determine the TENSAR Geogrid Force Coefficient, K, from Figure 4 using the values of slope angle, β, and factored soil friction angle, ϕ'_f.

4. Calculate the value of the total required tensile reinforcement force per running foot of slope, T (lb/ft), to be provided by all the geogrid layers:

$$T = \tfrac{1}{2} (\gamma) (K) (H')^2$$

5. Calculate the minimum required number of TENSAR Geogrids, N_{min}. Use a value of the long-term allowable design tensile strength, T_A.

$$N_{min} = T/T_A$$

6. Calculate the maximum allowable vertical geogrid spacing in the bottom of the slope, v (ft).

$$v = .6 (H') / N_{min}$$

If v is less than the minimum acceptable vertical spacing, s_{min}, a stronger geogrid is needed. Return to step 5.

If v is more than 3 to 4 feet, then a lighter duty geogrid should be used. Return to Step 5." (TENSAR, 1988)

The step following the number of TENSAR geogrid layers was to determine the length of each geogrid layer. The procedure for determining the length also came from the TENSAR design guide. The process is as follows:

"1. Determine the required TENSAR Geogrid length at the top of the slope, L_T (ft), and at the bottom of the slope, L_B (ft), using Figure 5A or 5B and multiplying by H'. The values of slope angle, β, and factored soil friction angle, ϕ'_f, are used with these figures.

2. If the required reinforcement lengths at the top and bottom of the slope are equal, i.e. $L_T = L_B$, then the length of geogrid layers will be the same and omit Step 3." (TENSAR, 1988)

Per the guidance in step 2 of determining the length of geogrid layers, the third step was omitted because the layer length on the top equaled the layer length on the bottom.

Following the criteria listed above and making assumptions about the soil's characteristics in the field, the design for the reinforced earth wall in the field was calculated to be:

Assumptions: $\phi = 32°$, $\gamma = 110$ lb/ft^3, c = 0 lb/ft^2

Design Criteria: Height = 14'
Front Slope = 45°
Rear Slope ≅ 80°
End Faces ≅ 80°
Safety Factor = 2.0

Solution:

<u>Required Number and Vertical Spacing of TENSAR Geogrids</u>:

1. $H' = H + q/\gamma = 14 + 0/110 = 14'$
2. $\phi'_f = \tan^{-1}(\tan\phi'/FS) = \tan^{-1}(\tan 32°/2) = 17.35° \cong 17°$
3. Determine Force Coefficient from ϕ'_f and slope angle

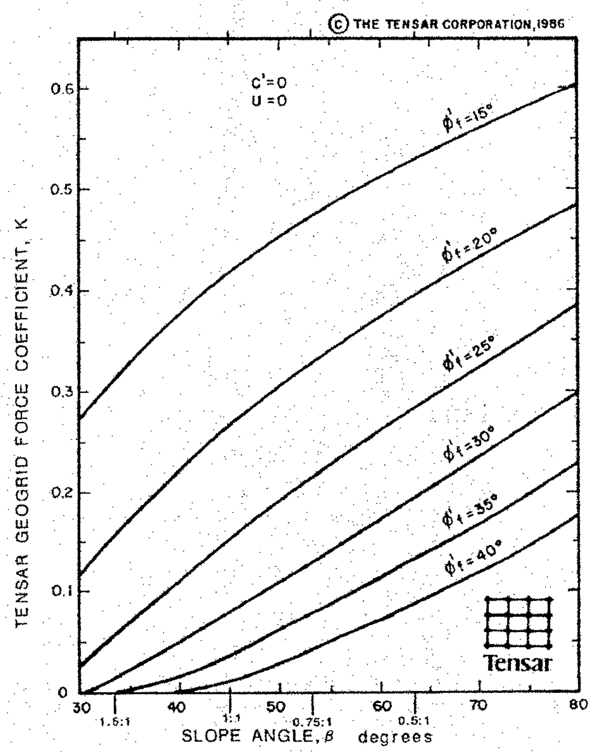


FIGURE 4

TENSAR GEOGRID FORCE COEFFICIENT CHART

Figure 3.1: TENSAR Geogrid Force Coefficient Chart

4. $T = ½ (\gamma)(K)(H')^2 = ½ (110)(.55)(14)^2 = 5929$ lb/ft
5. $N_{min} = T/T_A =$ Nmin = 5929/2000 = 2.96 ≅ 3 (UX 1400HS)

   Nmin = 5929/3700 = 1.6 (UX 1500HS)

6. $v = .6\,(H') / N_{min} = .6(14) / 3 = 2.8 \cong 3'$ (UX 1400HS)

$.6(14) / 1.6 = 5.25'$ (UX 1500HS)

*At this point the UX 1400HS Geogrid was selected because three-foot lifts seemed to be more easily constructed than five-foot lifts.

<u>Required Length of TENSAR Geogrids</u>:

1. From Figure 5A, TENSAR Geogrid Length Chart, the length of reinforcement was determined.

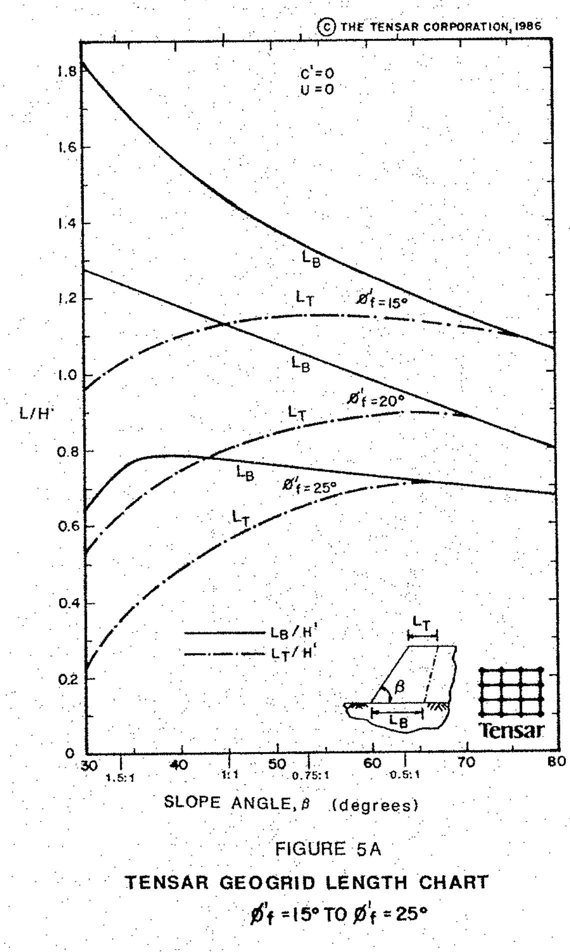


Figure 3.2: TENSAR Geogrid Length Chart

Using the slope angle, β, and the factored friction angle, ϕ'_f, the reinforcement lengths on the top and bottom of the earth wall were determined. The chart gives a L/H' value of approximately 0.95. From this relationship, where L/H' = .95 and H'= 14, the length of the geogrid strip is 13.3 feet. However, since there are going to be multiple primary reinforcement layers, the geogrid length was reduced to 12 feet.

Following the design for the reinforced earth wall, the next step was to check the safety factor using another method. Although the TENSAR Design was based on a safety factor of 2.0, the method is empirical and should be checked. The computer program Reinforced Slope Stability (RSS) was used for the analysis (See Appendix for the charts and the values that were generated). The advantage of using RSS is that it allowed the user to input the number of geogrid layers along with the geogrid's properties. The inputs placed into the program were made to model the reinforced earth wall constructed in the field. The following parameters were used:

Soil Properties:

Friction angle	32°
Cohesion	0 psf
Unit weight	110 pcf
Height	14'
Slope	80°
Internal SF	1.5

Reinforcement Properties:

Ultimate strength @ 5% strain	2000 lb/ft
Minimum embedment length	4'
Vertical spacing	3'
Number of layers	4

The default values were used for the reduction factor, the slope coefficient of friction, the foundation coefficient of friction and the embedded scale factor. In the help section of the program, default values are given for geotextiles versus metal strips or welded wire fabric. In all cases, the default value was checked with the help section so that the proper value was obtained. The safety factors generated using the reinforcement analysis method are as follows:

Most Critical Factor of Safety	1.528
Critical Zone in the Bottom	1.451
Critical Zone in the Middle	1.702
Critical Zone in the Top	2.237

The two values that were the most significant were the "Most Critical @ 1.528" and the "Critical Zone in the Bottom @ 1.451". Although they are less than the factor of safety used in the TENSAR Design, which was 2.0, they are close to the minimum design value of 1.5 given in the TENSAR Manual. Between the empirical TENSAR Design Method and using the RSS program for a safety factor check, the design seemed to be competent. The biggest challenge was applying the design to the field.

SECTION IV

SOIL DESCRIPTION AND TESTING

A. OBJECTIVE

The purpose of this section is to classify the soil that was used for constructing the reinforced earth wall. Several assumptions were made when using the TENSAR Design Method including the friction angle and the cohesion of the soil. Specifically, the friction angle was assumed to be 32° and the cohesive value was 0 psf. The purpose of the testing is to validate whether or not the assumptions were close to the actual values in the field.

B. SOIL DESCRIPTION

The soil was stockpiled at White Sands Missile Range for the construction. During construction, the temperature ranged from 90° F during the day down to 60° F during the night. There was no rainfall during the construction period and the climate was arid. The moisture content of the soil was not determined.

C. SOIL TESTING

The tests performed on the soil were Consolidated Drained Triaxial Tests and a Sieve Analysis that followed ASTM D 421 (Liu and Evett, 1990).

Triaxial Tests:

The triaxial tests were performed following ASTM D 2850-87 (Liu and Evett, 1990). The sample was dry and was compacted with a metal rod during the sample build-up. The chamber pressure was applied through air only. Water was not used. A vertical

load was applied fairly rapidly, .15 mm/min and the sample was drained. The test was run with increasing load, 0.50 mm/min, until the sample failed. At this point, the maximum load was recorded and the deviator stress at failure was calculated. The specimen and triaxial data are listed in the following table:

Table 4.1: Specimen and Triaxial Data

Test #	Diameter (in)	Height (in)	Area (in^2)	Chamber Pressure σ_3, (psi)	Deviator Stress σ_d, (psi)	Major Principal Stress, σ_1, (psi)
1	2.80	5.53	6.157	14	62.6	76.6
2	2.78	5.44	6.04	28	126.76	154.8

See Appendix B for the data collected during each test. Since the values only needed to be approximate, neither the height correction nor the area correction was calculated. Instead, the maximum load was divided by the initial area to determine the deviator stress. Then, the deviator stress was added to the minor principal stress (chamber pressure) to calculate the major principal stress. Using the minor and the major principal stress from each test, two circles were plotted to give the Mohr-Coulomb failure criteria.

From the two unsaturated triaxial tests, the angle of internal friction was determined. Since the sample was an unsaturated medium sand, the cohesion value was zero, as expected. The next plot shows the Mohr-Coulomb failure envelope and gives the friction angle and cohesion value.

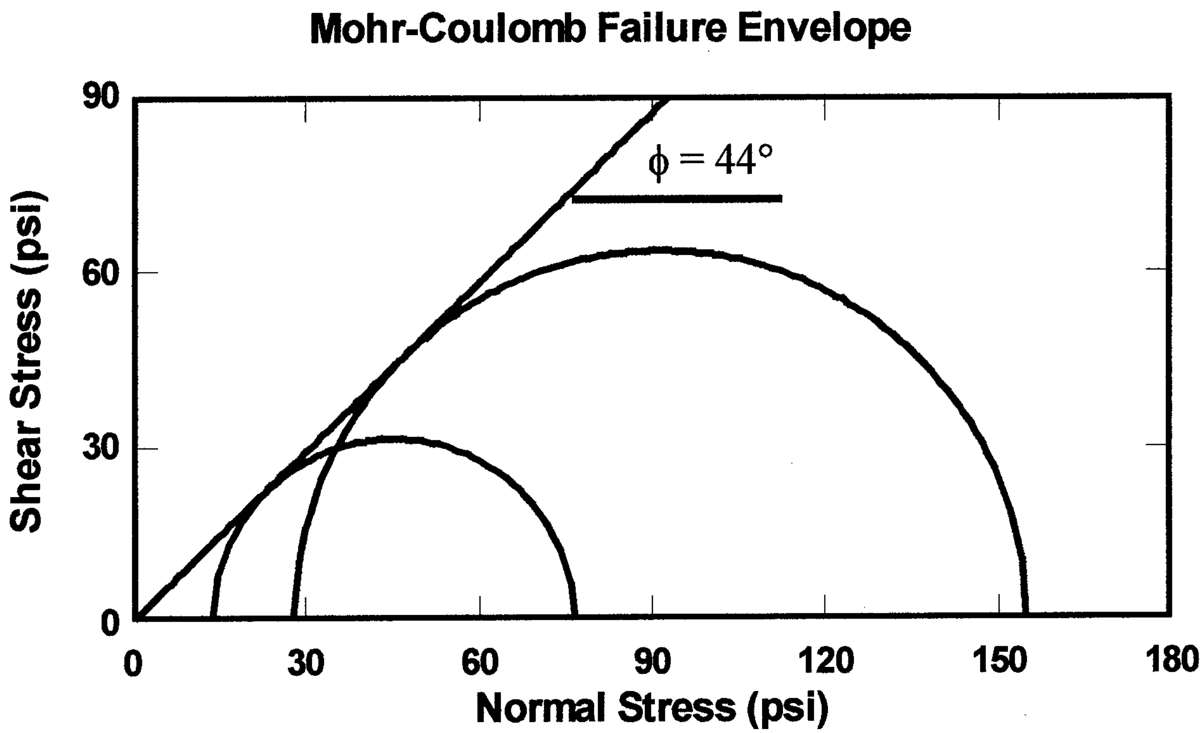


Figure 4.1: Mohr-Coulomb Failure Envelope

This plot shows a high friction angle, 44°, for a medium sand and a cohesion value of 0. For a medium sand, the expected friction angle range is 26°- 30° (Lambe and Whitman, 1969). Comparing the tested values to the expected values, there is a large discrepancy between the two. Although the test results show a friction angle of 44°, this value is very high and a more reasonable value of 32° would be used. Also, if the design values were necessary for future construction, further soil testing would be accomplished. However, since the testing was only intended to validate the design values already used, only two tests were performed.

Sieve Analysis:

The soil was sieved through six sieves to produce the sieve analysis plot. Numbers 4, 10, 20, 40, 100, 200 were used. From the plot, the Unified Soil Classification System was used to characterize the soil. The soil was a poorly graded sand.

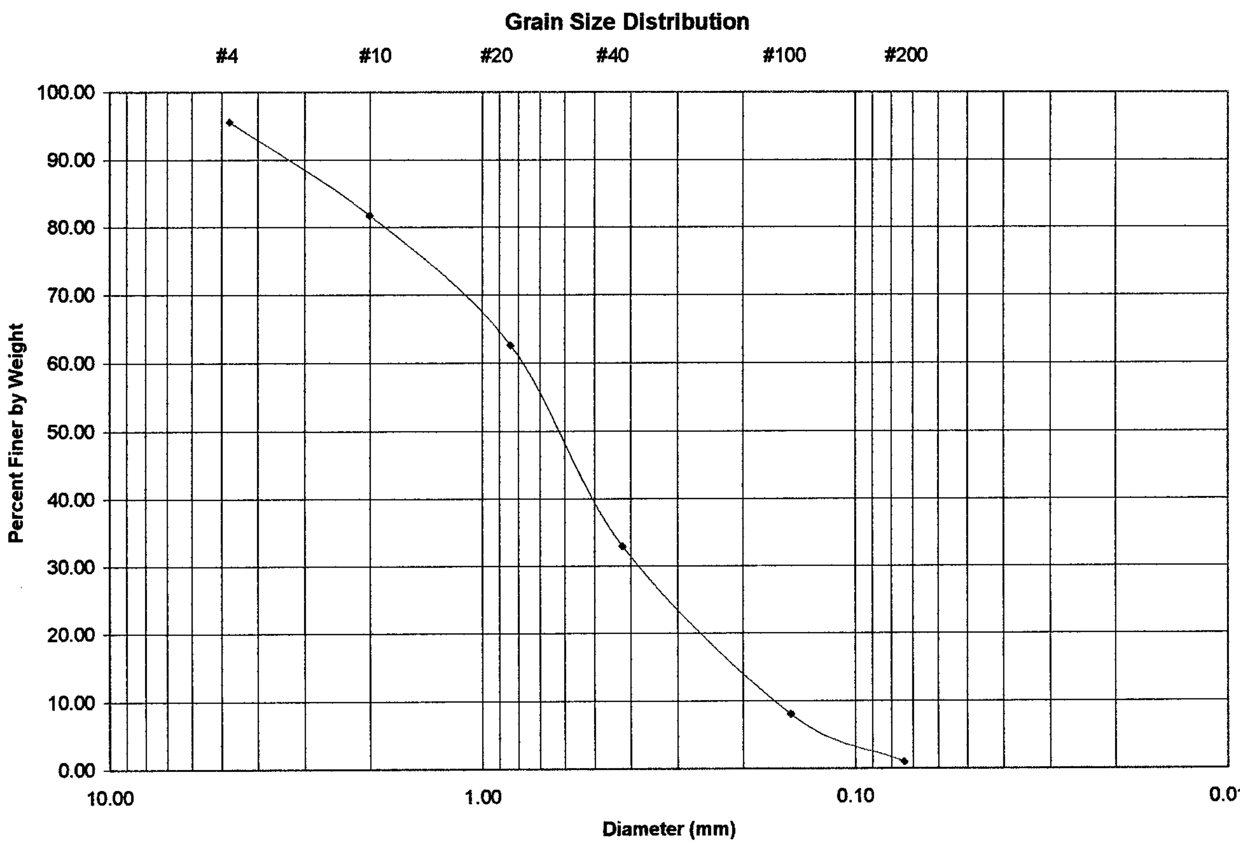


Figure 4.2: Grain Size Distribution

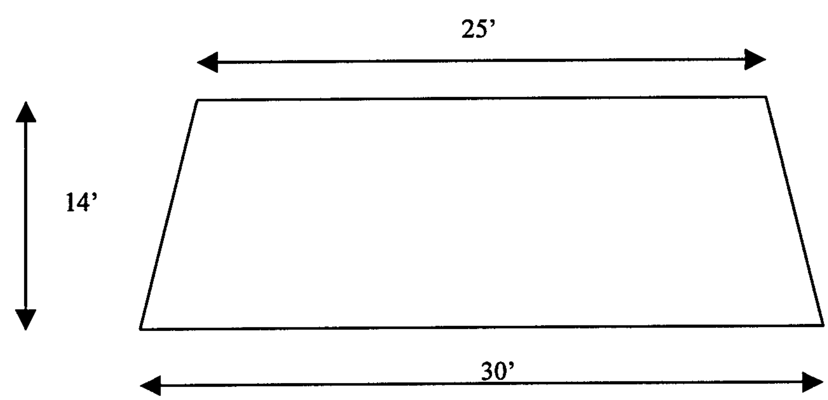
SECTION V

REINFORCED EARTH WALL CONSTRUCTION

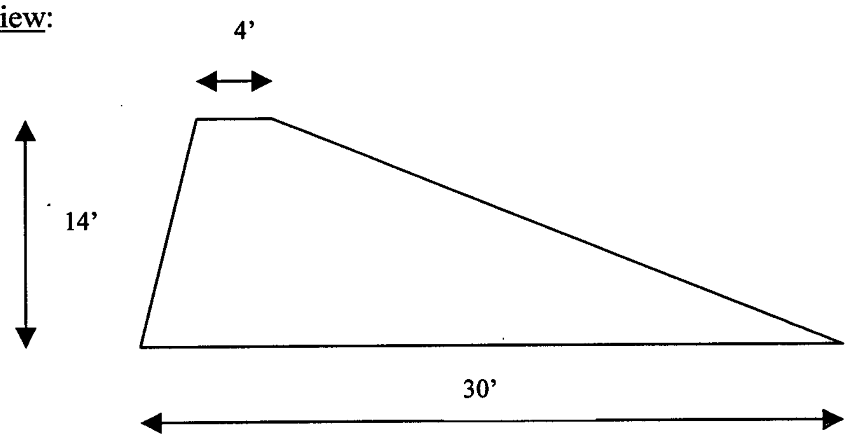
A. OBJECTIVE

The purpose of this section is to address the construction process, the equipment used, the time required, the soil used, design modification and the changes made in the field to compensate for the design changes. The reinforced earth wall that the author intended to construct had effective dimensions of 14 feet high, 25 feet wide and approximately 30 feet long from the rear face (80°) to the front face. The next diagrams will show the dimensions and shape of the reinforced earth wall.

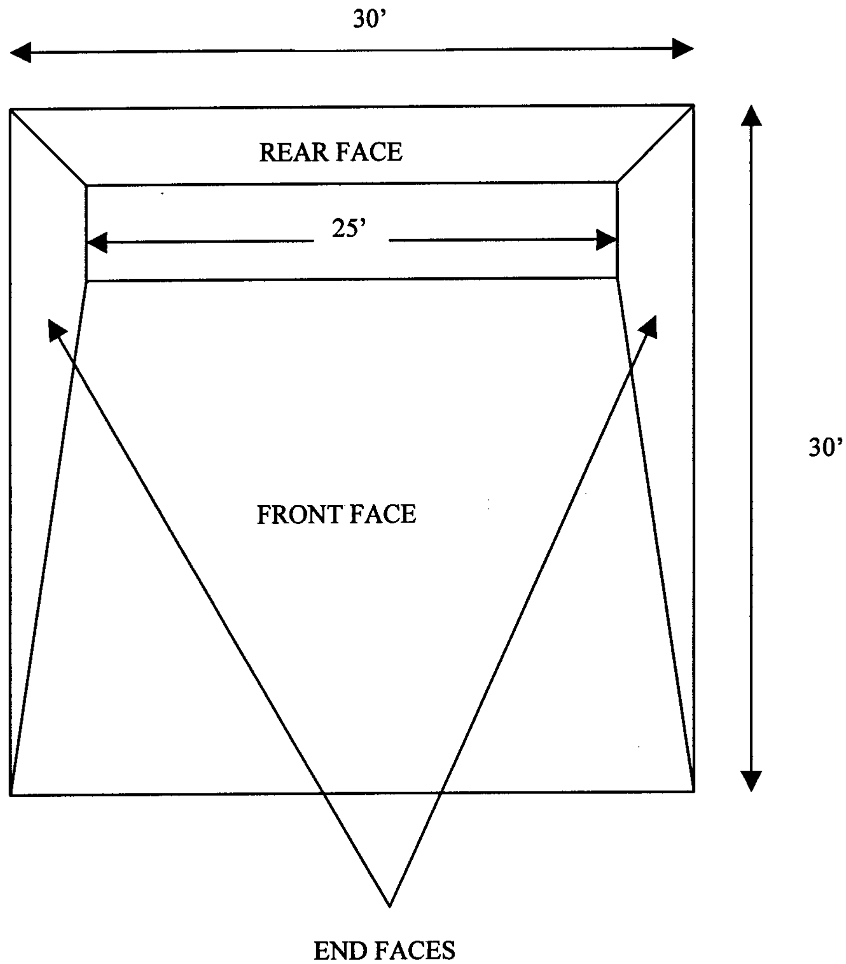
Front View:



SideView:



Plan View:



B. CONSTRUCTION

The construction took place at the White Sands Missile Range, NM. The range is located approximately 75 miles south of Albuquerque, NM. The actual site on the range is the High Explosive Testing Range, approximately 1.5 miles west of the Trinity Range. The site was cleared and grubbed prior to our arrival. The first step in the construction process was to level the site and locate the extents of the reinforced earth wall. Surveying was provided by PMR Construction. The site was surveyed to reflect the test layout provided in Figure 1.2.

After the layout was completed, the geogrid and erosion mat materials were separated for cutting and assembly. The uniaxial geogrid (primary) and biaxial geogrid (wrapping material) arrived to the site in rolls. The rolls' sizes were:

uniaxial (UX1400HS)	width = 4.26'
	length = 251.5'
biaxial (BX1220)	width = 9.8'
	length = 164'
erosion mat	width = 10'
	length = 150'

From these dimensions, the geogrid and erosion mat materials had to be cut and assembled differently. The uniaxial geogrid was cut and assembled two different ways. The first group was cut into twelve-foot sections to provide the reinforcement for the rear face. The geogrid started at the rear slope and was placed twelve-feet in the direction of the front face. Each section was overlapped between 3-6 inches. The second group was cut twenty-five-feet and placed from one end to the other end. It was also overlapped between 3-6 inches. The erosion mat was attached to the biaxial geogrid and used as a geotextile. It prevented the soil from flowing through the spaces between the biaxial geogrid. The actual placement and methods of attachment will be discussed from the bottom lift to the top of the earth wall. There was not one particular routine since each lift varied significantly. The method was supposed to be similar for each lift, but due to problems and oversights, it was not.

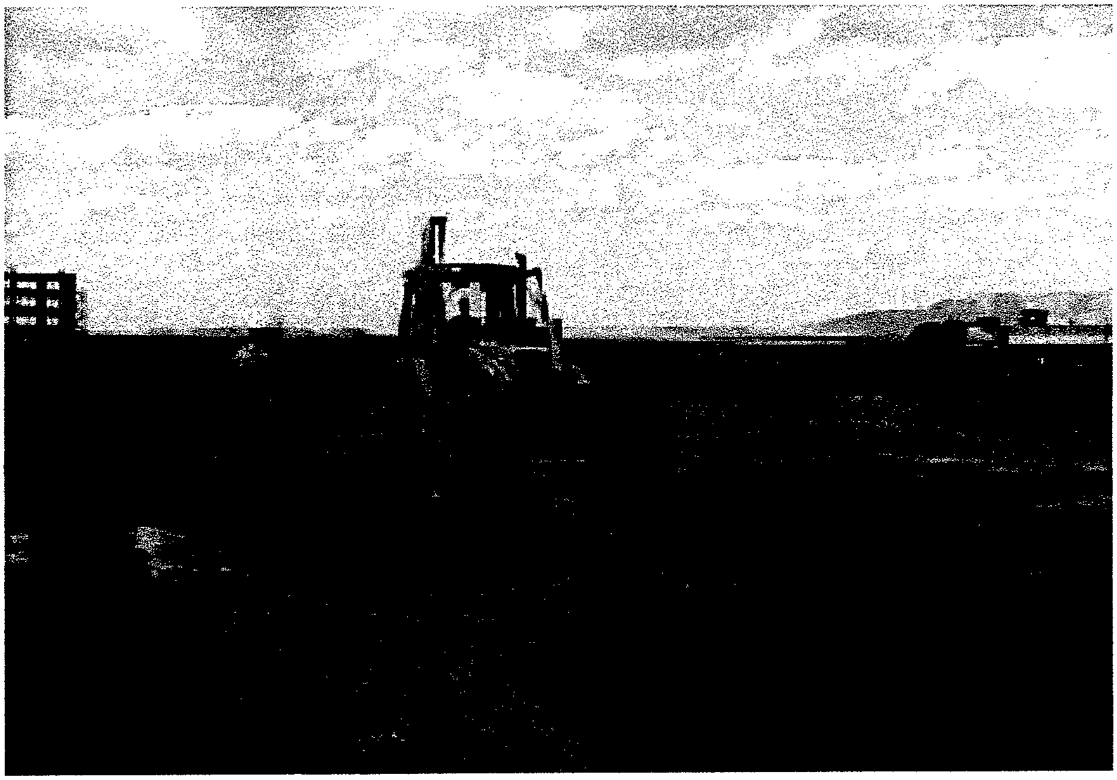
Lift # 1:

The first lift was constructed fairly easily and per the design. The ease of construction is attributed to free movement of the construction equipment since the construction was at the ground level. The first step was to attach the erosion mat to the biaxial geogrid material. The erosion mat was attached to the biaxial geogrid using duct tape and wire ties. The duct tape allowed the crew to fasten the erosion mat temporarily so the wind would not blow it away. The wire ties were punched through the erosion mat and twisted around the ribs of the biaxial geogrid. The ties were fastened every twelve inches. Since each lift's height was approximately three feet, the erosion mat needed to cover the middle six feet of the biaxial geogrid. This provided the three-foot coverage where the soil would flow through the geogrid and allowed one and a half feet of overlap on the top and the bottom of the geogrid. Since some bulging was anticipated, the overlap was necessary to keep the soil inside of the erosion mat.

After attaching the erosion mat to the biaxial geogrid, the next step was to position the geogrid. The shape looked like a horseshoe since the initial plan was to have three 80° slopes. The geogrid was positioned so that four feet of the biaxial geogrid would rest beneath the first lift.

Photograph 5.1: Placing Biaxial Geogrid

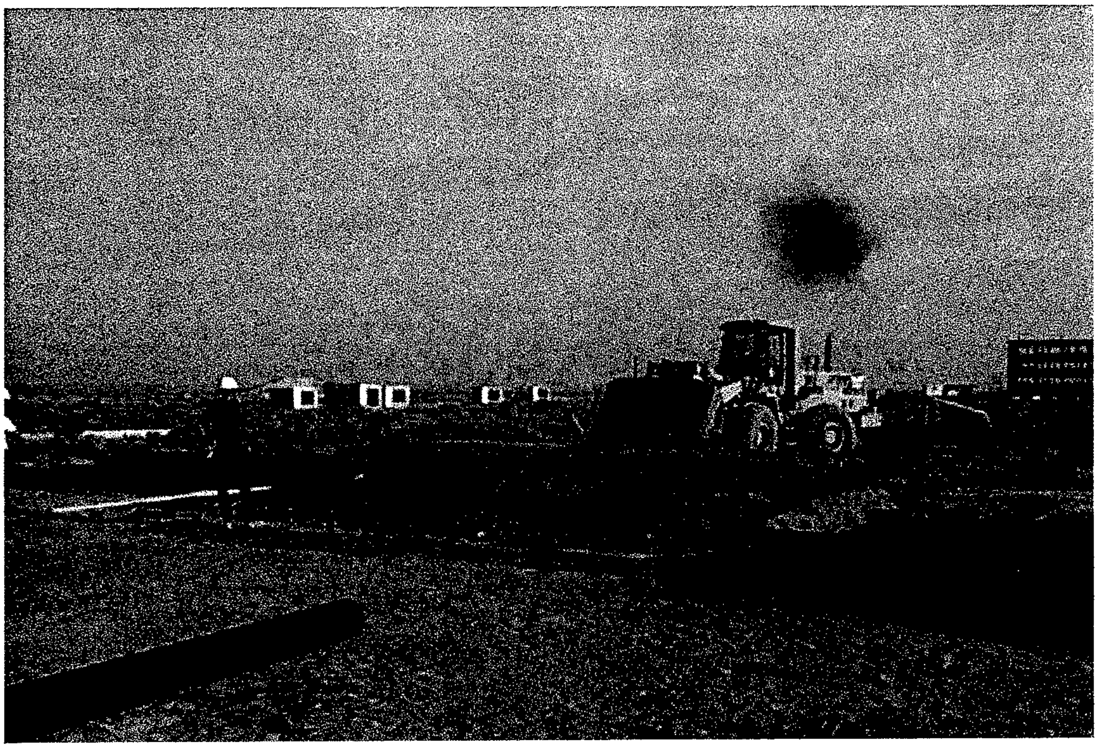
The rest of the geogrid was laid flat on the ground. The next step was to bring in one foot of fill for the first uniaxial geogrid layer. A loader with a three-cubic-yard bucket dumped the fill and a backhoe worked the site to spread the soil evenly. When the fill was level at sixteen inches, the loader worked the lift to gain some compaction. Following two passes, the level was about ten inches in elevation. The loader brought more fill and the backhoe worked the soil until the lift was level at twelve inches.



Photograph 5.2: Placing and Leveling Soil for Lift #1

At this point, the first uniaxial geogrid was placed. The geogrid was placed at the rear wall and rolled twelve feet toward the front face. To keep the geogrid from rolling back to the rear wall, shovels were used to dump soil on top of the geogrid. The next step was to bring the lift up another twelve inches so the uniaxial geogrid could be placed from one end to the other end. The loader and backhoe worked in tandem as they had during the first twelve inches until the height reached twenty-four inches. Once twenty-four inches of elevation had been reached, the procedure was changed. Since the entire

width of the wall was twenty-five feet from end to end, the entire section with the uniaxial geogrid rather than cutting two, twelve-foot sections. The drawback of using extra material was accepted since it sped the process of placing the geogrid.



Photograph 5.3: Placing and Leveling Soil for Lift #1

The next step was to place the last twelve inches of fill so that the first lift could be wrapped with the biaxial geogrid. The loader system was repeated until the lift reached three-feet in elevation. The lift was brought to approximately 40" in elevation. Next, the loader compacted most of the area but stayed away from the edges.

Photograph 5.4: Compacting Lift #1

However, at this point the first problem was encountered. Since there were no wire baskets to retain the shape of the outside edges, the soil had fallen along the edges at about 45°. Before the sides could be wrapped with the biaxial geogrid onto the top of the lift, the edges had to be reshaped so that they were as vertical as possible. This problem was partially overcome using shovels and the skid-loader. Using the bucket on the skid-loader, most of the fallen soil was removed from the erosion mat. Then, shovels were used to pack the bottom twelve inches. The method was not perfect but it allowed the crew to remove most of the loose soil that rested on top of the erosion mat.

Finally, the biaxial geogrid was wrapped around the rear face. With four men on the ground level lifting the geogrid to the four men standing on the three-foot lift, the

geogrid was pulled on top of the first lift. To keep the biaxial geogrid in place, the skid loader dumped a load of soil on top of the fabric. The same process was performed on the ends of the reinforced earth wall. Another problem that existed was the corners where the biaxial geogrid was folded. For the three-foot lift at the corners, wire ties were used to tie the fabric together. The wire ties were placed every three inches so that the corners did not bulge.

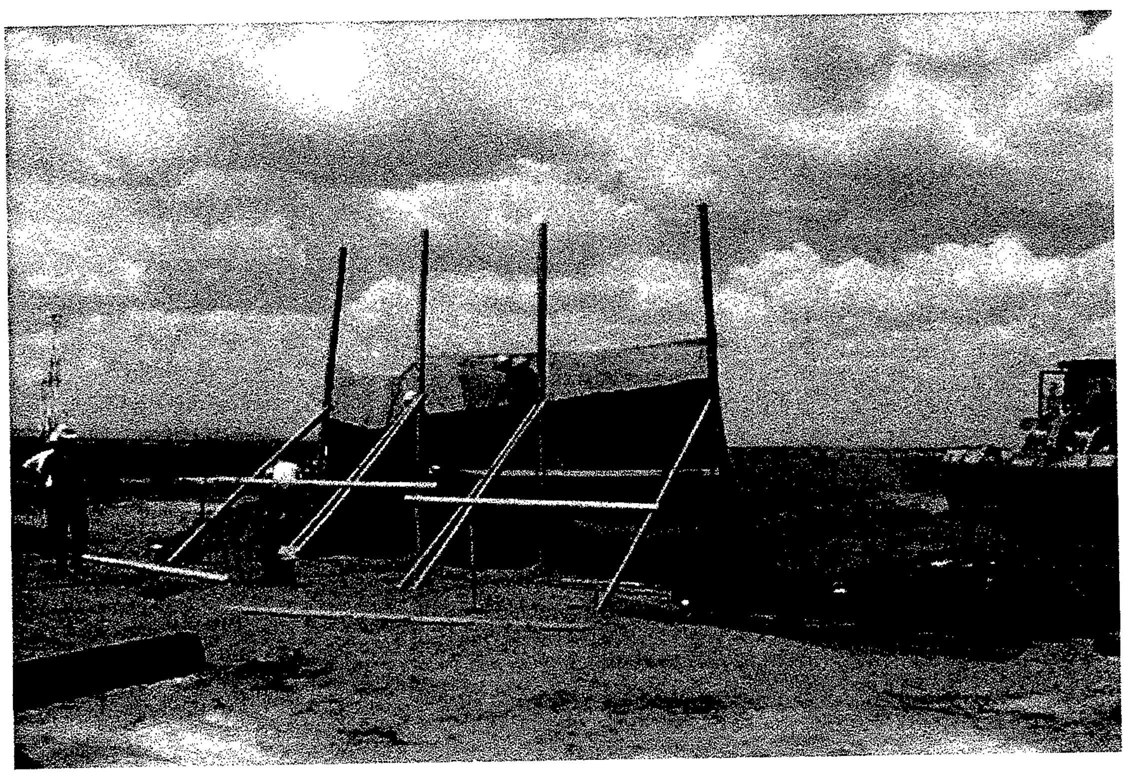
Lift #2:

In order to move the equipment and the soil onto the second lift, the front face of the earth wall was sloped to the ground elevation. The construction of the second lift was supposed to be identical to that of the first lift. It began by cutting and tying the biaxial geogrid. Next, the erosion mat was attached inside the biaxial geogrid, the fabric was spread out and the first twelve inches of soil was placed. After placing and compacting the first twelve inches, the primary geogrid was rolled from the rear face to the front face. Next, another twelve-inch lift was placed and compacted and the primary geogrid was run across the wall from end to end. Finally, another twelve inches was placed and there was an attempt to pull the biaxial geogrid wrap around the second lift. The problem was that the soil could not be kept from falling at the edges. The same method that was used in the first lift was tried but the results were unsuccessful. Finally, the biaxial geogrid was pulled around the edges but the desired angle of 80° could not be attained. The wrap bulged much more than the first lift and there were large voids where the soil had collapsed into the bottom of the second lift.

At this point, the design was altered so that a blast mitigating wall could still be constructed for the test. Because of the bulging that occurred within the biaxial geogrid and the limited time available for construction, the concept of having the rear face and the end walls sloped at 80° was abandoned. Instead, all the efforts were directed into the construction of the rear face near 80°. A form wall was constructed using lumber that was on-site. The wall consisted of four 4" x 4" x 18' posts, three 4' x 8' sheets of plywood and many 2" x 4" and 2" x 6" braces. The posts were embedded approximately twelve inches into the soil and they were spaced eight feet apart. After the posts were

positioned, the braces were attached to the posts in two different ways. First, the braces were nailed across the posts. Second, the braces were nailed eight feet up the posts and angled back to the ground. Stakes were then hammered at the base of the braces to keep them from moving. The form wall was not positioned at exactly 80° but it was angled nearly vertical and allowed construction to continue.

The next step was to construct a moveable wall panel to lean against the 4" x 4" posts. To do this, more 2" x 6" studs and plywood sheets were used. The lumber was nailed to the plywood such that the lumber would rest between the posts and the plywood. This left a smooth plywood face that that the third lift was to be constructed against.



Photograph 5.5: Form Wall for Reinforced Earth Wall

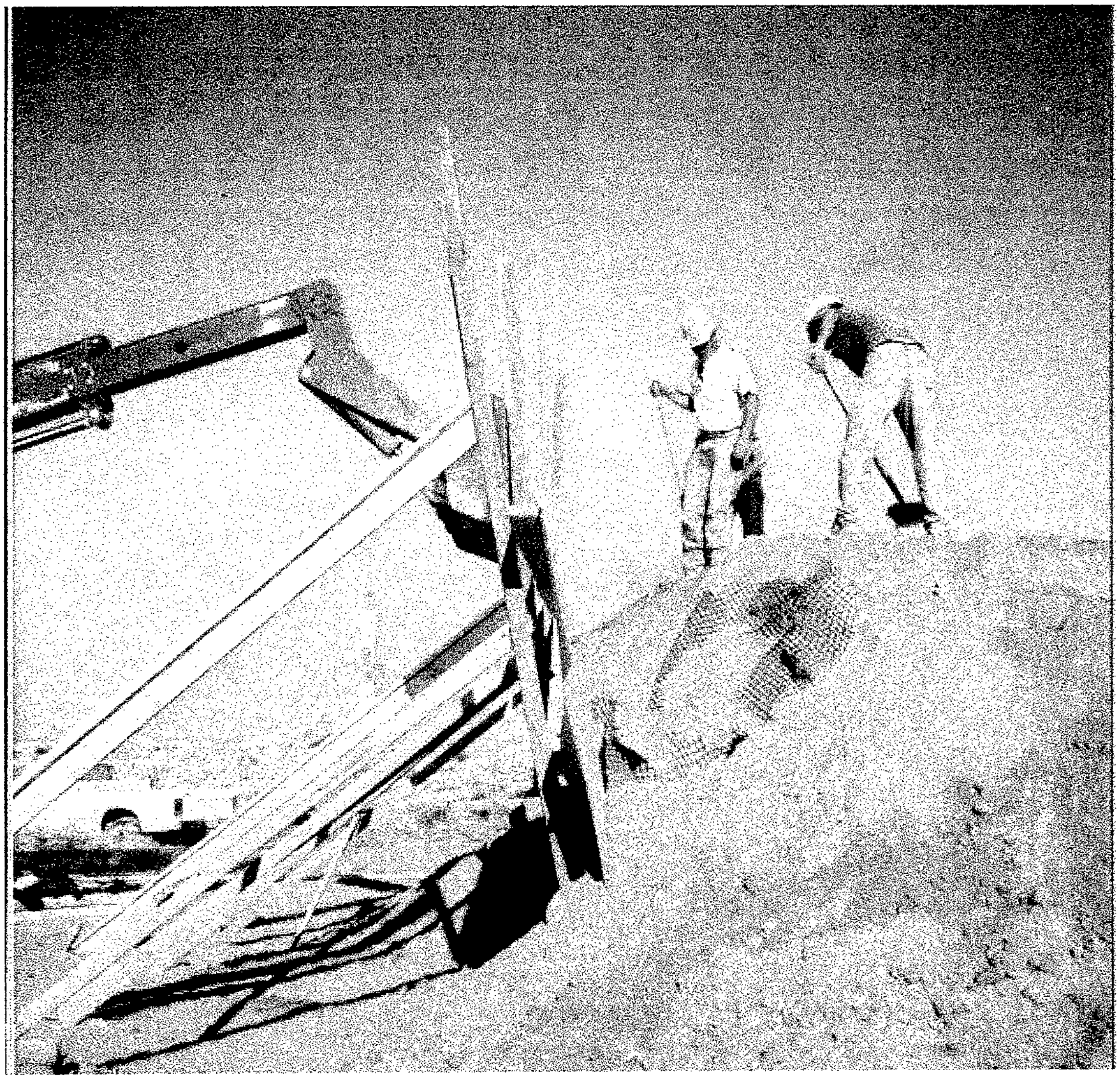
Prior to starting the third lift, the end faces of the reinforced earth wall were built-up. This had to be done because the twenty-five foot span that was supposed to be constructed was gradually narrowing. The narrowing was attributed to our inability to construct the end walls at 80°. There was not enough lumber to construct a form on the end walls so they were built-up on the sides using additional soil. After this was completed, the angle of the end walls was approximately 35°. Once the end walls were built-up so that the loader could continue adding soil to the entire twenty-five foot width span, the third lift began.

Lift #3:

This lift started at an elevation of six feet and had significant changes from the design. Most obvious was that there were three faces, the front and both ends, that were sloped at approximately 30°. Also, the rear face was constructed against a temporary wood form so an 80° face could be constructed. The method of geotextile placement also changed since only one face was being reinforced. With the field change, only one layer of primary reinforcement was used and the biaxial geogrid was only placed to wrap the rear face. These changes sped the construction process because the amount of handwork decreased and the number of lift increments dropped from three lifts to two lifts.

The first step was cutting and placing the biaxial geogrid to form the third lift. After placing the biaxial geogrid so that it had four feet of cover, the remaining geogrid section was run up the plywood and temporarily tacked into the 4" by 4" posts. The second step was placing and compacting one foot of fill prior to placing the primary geogrid. After the primary geogrid was set, the remaining two feet of soil was placed to form lift three. However, just before the lift approached the three-foot point, the final disaster surfaced.

As the loader dumped one of its three cubic yard loads, one of the end posts cracked and the form wall deflected about two and a half feet.



Photograph 5.6: Collapsed Form Wall for Reinforced Earth Wall

This caused several problems. The first and most apparent was that the wall's angle increased from 80° to about 100° in the section of the form where the post had cracked. In addition, the other posts were cracking due to the increased stress that resulted from the failed post. In order to salvage this lift before the other posts failed, the lift was finished even though it was not quite three feet high. However, the form wall failure caused the same problem that was supposed to be avoided. Instead of the form

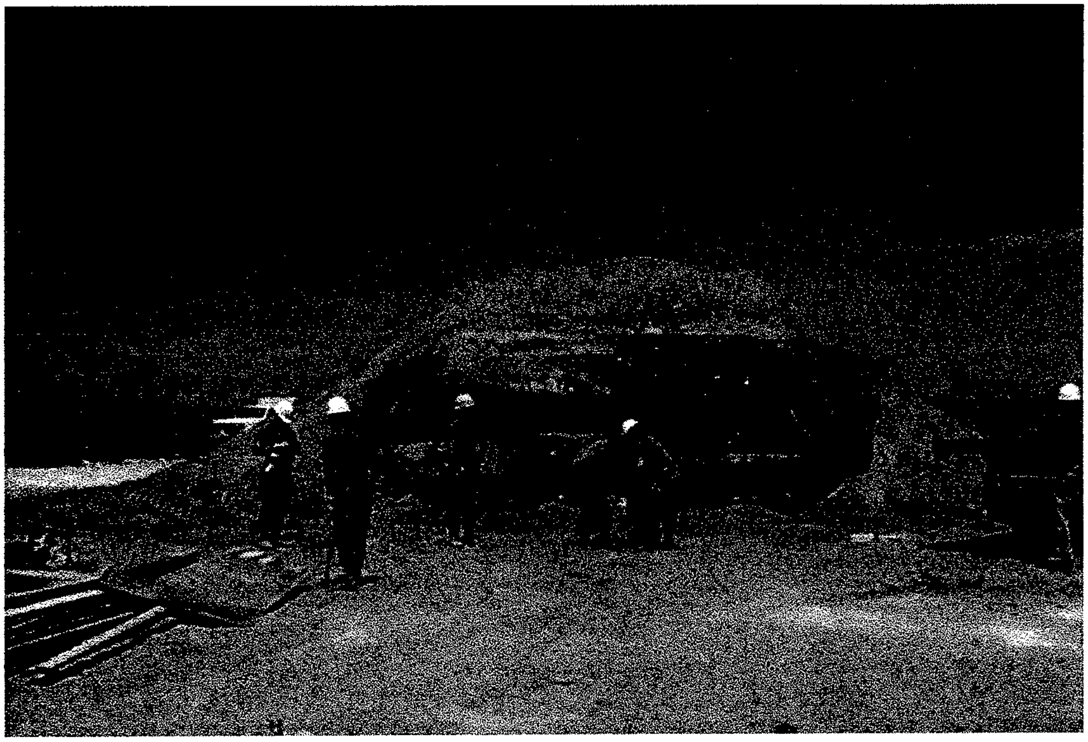
wall providing the temporary 80° angle and keeping the soil confined against the plywood, when the form wall failed, the soil fell at its natural angle of repose. Once again, this resulted in an excess amount of soil at the bottom of the lift, and a void towards the top of the lift. Also, the excess soil at the bottom caused a bulge and decreased the amount of biaxial geogrid that would be available on top of the lift. Since each lift was dependent upon the biaxial geogrid to hold the soil inside the lift, this was a significant problem. By the time the biaxial geogrid was wrapped around the lift, there was only two feet of the material left for the top of the lift. This was not enough overlap to prevent the biaxial geogrid from "pull-out failure" so six-foot sections of the geogrid were cut and spliced to the existing geogrid. The splice was overlapped approximately six inches and wire ties were fastened every three inches. After this was completed, soil was placed and spread on top of the geogrid so that the biaxial geogrid section would not pull-out if the form wall collapsed.

The next decision involved how to meet our objective after all the problems that were encountered. After discussing the situation with the test sponsor and the construction superintendent, it was decided to salvage the existing structure but stop the reinforced earth construction. The major factor that drove this decision was time. To continue the reinforced earth construction, the contractor would have required at least three more days. This would have allotted one day to construct and properly brace a large form wall and two days to finish the construction. However, there was only one day remaining to construct a testable wall. Measurements determined that the reinforced earth section was approximately nine feet high and that a near vertical face had been constructed in front of the general-purpose shelter. A decision was made that this barrier

would protect the structure against the blast effects but that more soil should be placed on top of the reinforced earth wall until a fourteen-foot high structure was attained.

After this decision was made, the construction went very quickly. The large loader piled soil on top of the third lift in a pyramidal manner. Soil was placed as close to the rear edge as possible. Since there was a concern that placing a surcharge on top of the third lift could potentially cause a slope failure, the soil was placed six-feet from the rear face. In addition, the fourteen-foot high barrier needed to be as wide as possible. The concern was that if the total width of the general-purpose structure was not protected all the way up to fourteen-feet, the blast wave could wrap-around the reinforced earth wall. Although every effort was made to construct the wall to the design width of twenty-five feet, the earlier design change made this impossible. Since the end walls weren't at the eighty-degree angle that was desired, the available area where soil could be placed had decreased. When the wall's height finally reached fourteen-feet, it was only at the design height for ten feet in width before it started sloping towards the ground.

The biggest question that evolved from the construction of this reinforced earth wall was what happened during the construction phase. Why was the construction of this wall so difficult?



Photograph 5.7: Completed Reinforced Earth Wall

This question was a huge concern since these walls need to be constructed quickly in a high-threat environment. First of all, the designed reinforced earth wall was not completed. Second, it took four days to produce a structure that was only partly reinforced. The construction of the wall raised some troubling questions about the ability to implement reinforced earth wall construction in expedient situations. This is because once the wall was completed, there was nothing that could be done to improve the wall's resistance to external and internal failures. After the soil and reinforcement were placed, only nature's laws and a little luck could keep the reinforced earth wall intact. Conversely, the appearance of the ugliest looking reinforced earth wall was directly attributed to me. Most of the blame can be related to my inexperience with constructing

reinforced earth walls. Although the literature and the design consultant from TENSAR clearly stated that wire baskets must be used when constructing reinforced earth walls over 45°, the author was not insistent with the agency that purchased the materials. The original design included wire baskets at a cost of $952 for the three reinforced sides (ends and rear). At the decision point, the total cost for the design was approximately $5,500. By eliminating the wire basket system, a savings of about $1000 could be realized. So, rather than insisting that the wire baskets were crucial during the construction process, and that they were absolutely necessary for constructability, they were dropped from the design. After seeing the resulting problems first hand, omitting the wire baskets became the fatal error for the reinforced earth wall construction.

SECTION VI

DIVINE BUFFALO V TEST

A. OBJECTIVE

The purpose of this section is to describe the test performed at White Sands Missile Range, NM. The test was performed on 23 September 1998, and was designed to test expediently constructed walls against blast effects. The three walls that will be compared include a Hesco-Bastion Concertainer Revetment Module wall, a Maccaferri FlexMac Revetment Module wall and a reinforced-earth wall using Tensar Geogrid Reinforcement. Also, this section will compare the instrumentation results and the pictures taken before and after the blast.

B. TEST RESULTS

The first section of discussion will cover the expected and recorded pressures from the blast. The pressure gauges were mounted in three positions with respect to the walls. They were placed in front of the walls, directly behind the walls between the wall and the shelter and inside the shelters. The following table will compare the differences between the general-purpose shelter behind the reinforced earth wall and the general-purpose shelter that was left unprotected. It will also compare the pressures on three temper tents. Two of the tents were protected from walls and the third was left in the open. All of the shelters and tents were placed two-hundred sixty-seven feet from the detonation of the C-4. Finally, a fourth tent located at three-hundred thirty feet from the detonation will be compared to the structures at two-hundred sixty-seven feet to show the advantage of having stand-off distance.

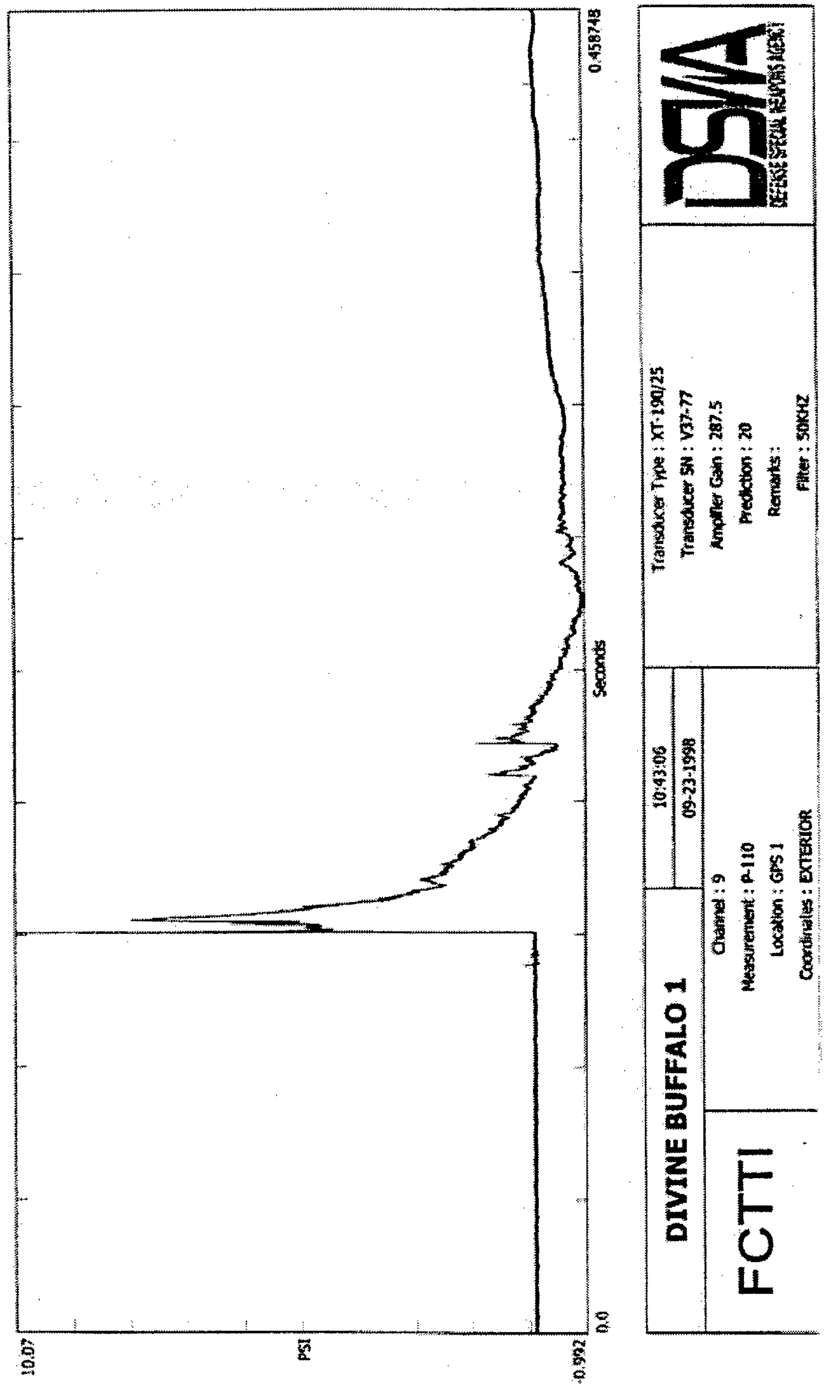


Figure 6.1: GPS 1 was an unprotected general-purpose shelter at 267' from the blast. The pressure transducer was located in front of the shelter.

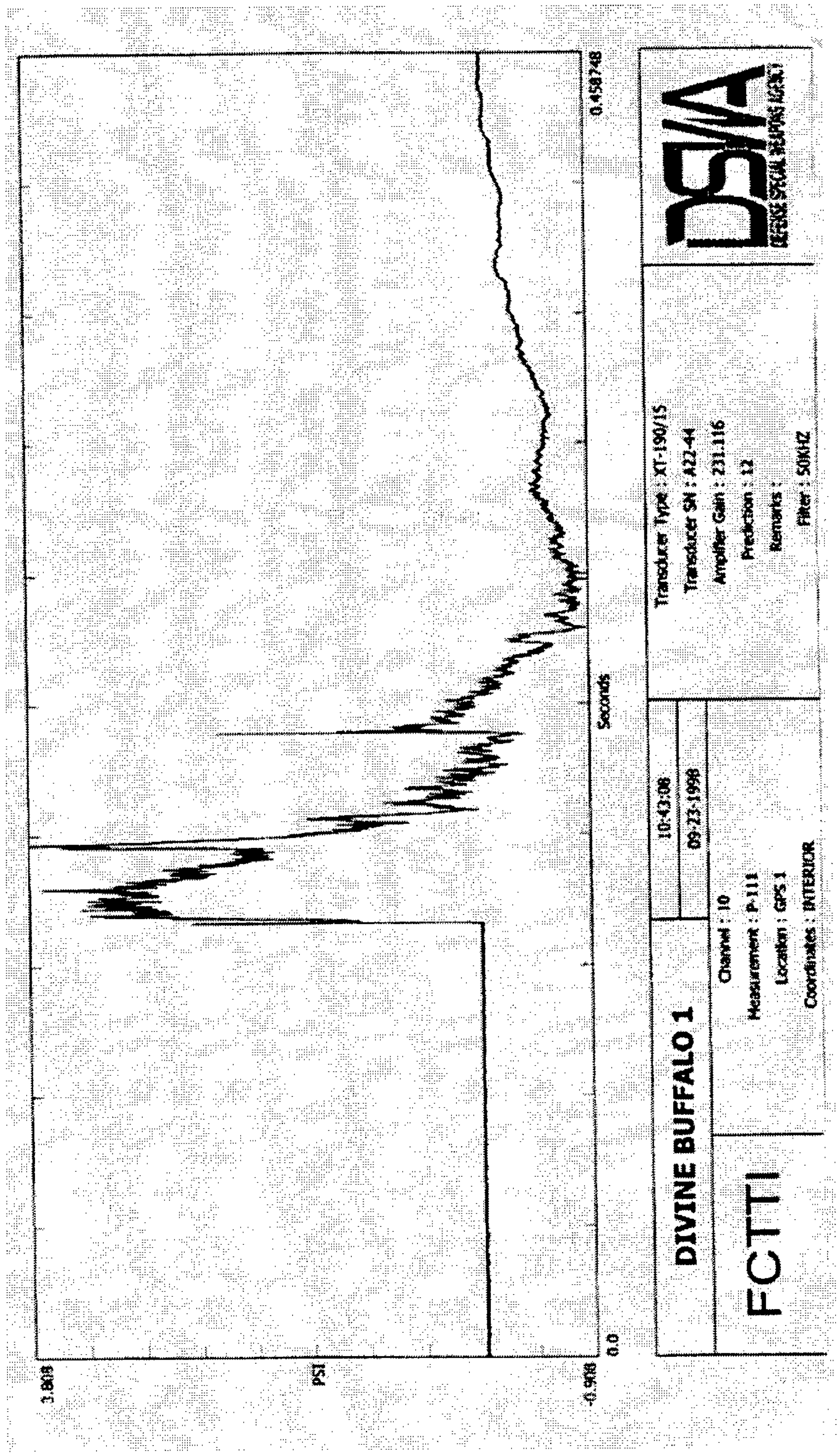


Figure 6.2: GPS 1 was an unprotected general-purpose shelter. The pressure transducer was located inside the shelter at 277' from the blast.

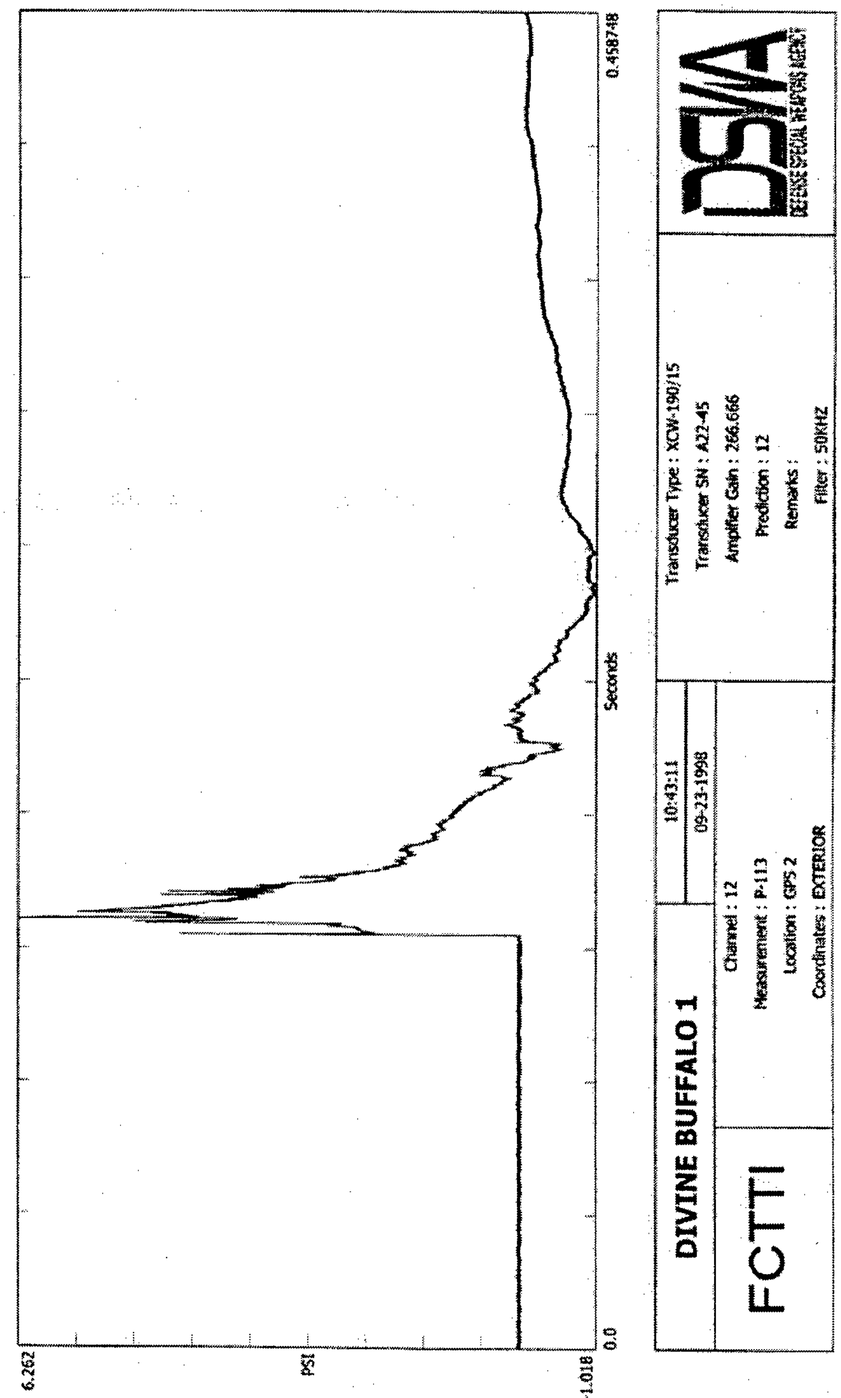


Figure 6.3: GPS 2 was a general-purpose shelter placed behind the reinforced earth berm. The pressure transducer was placed outside the shelter between the shelter and the wall and was located 270' from the blast.

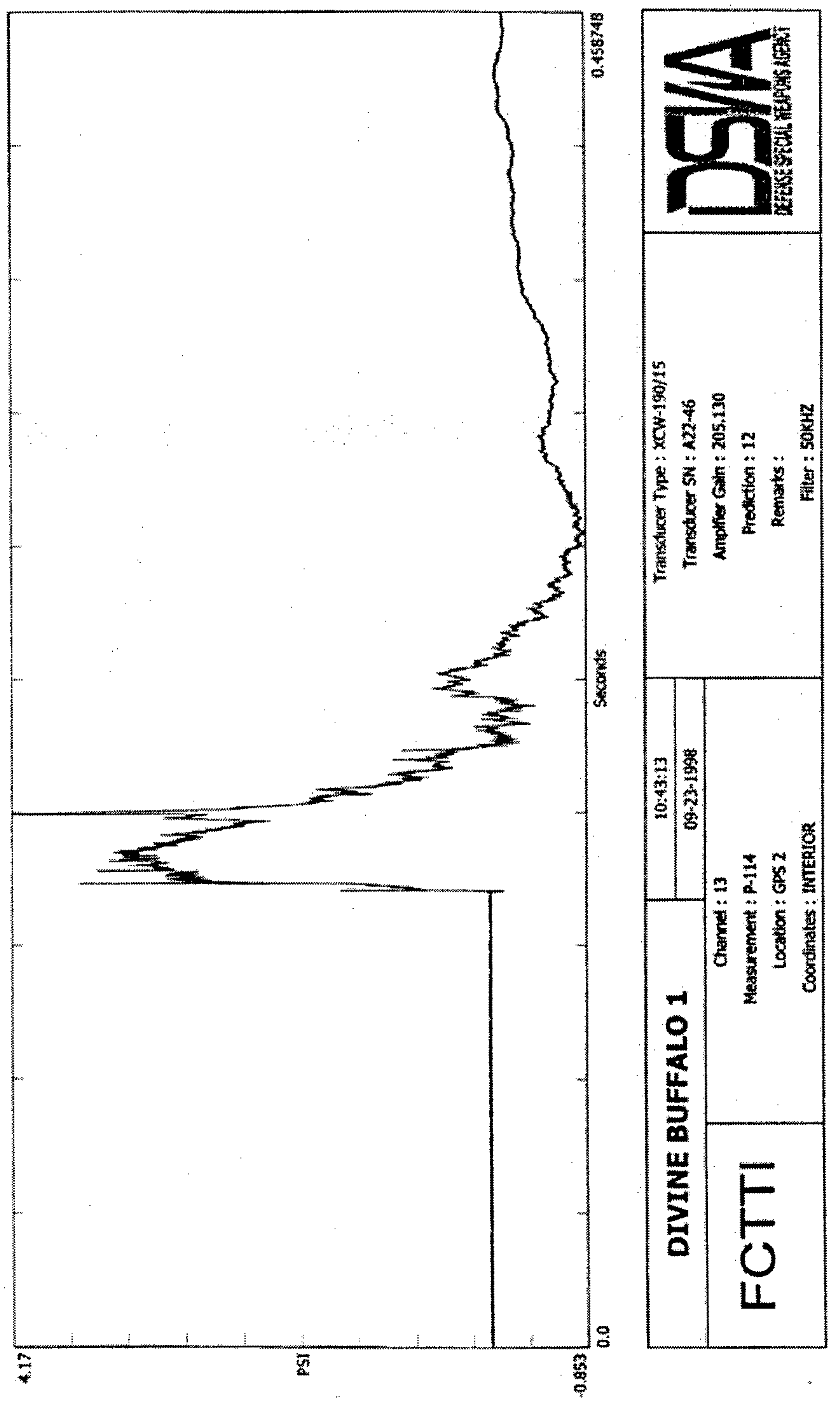


Figure 6.4: GPS 2 was a general-purpose shelter placed behind the reinforced earth wall. The pressure transducer was placed inside the shelter and was located 283' from the blast.

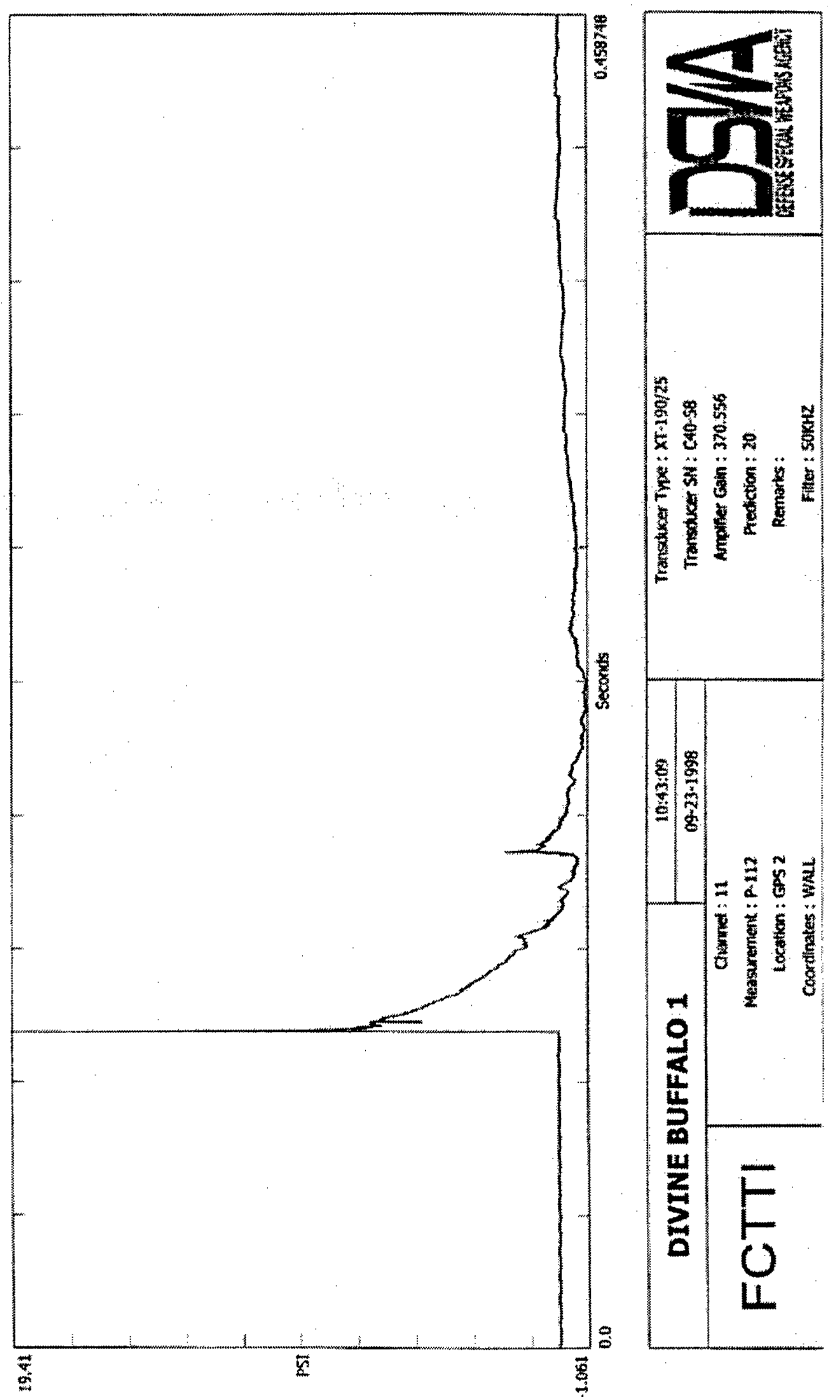


Figure 6.5: This pressure transducer was positioned on the front face of the reinforced earth wall approximately 4' above ground level. It was located about 250' from the blast.

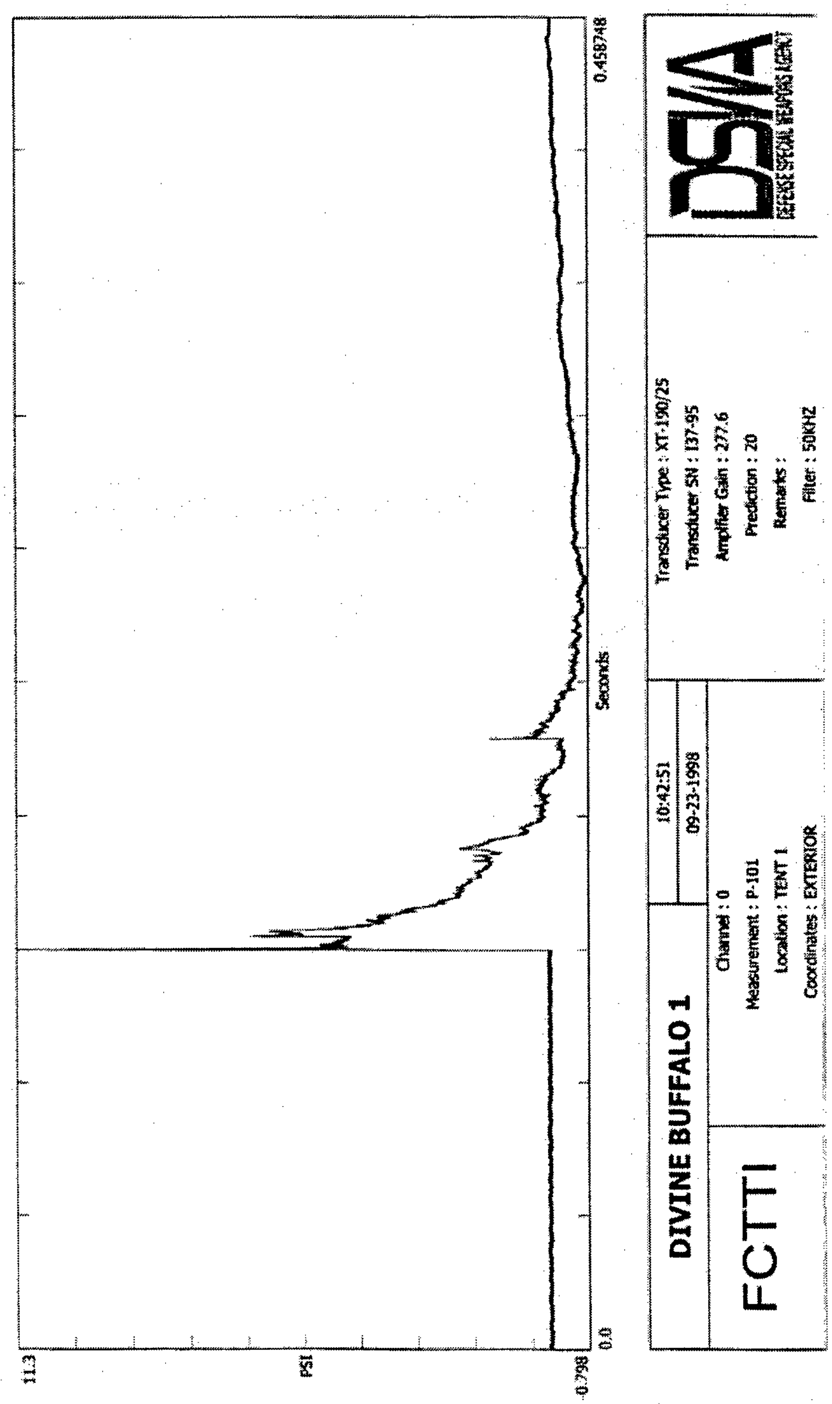


Figure 6.6: Tent 1 was unprotected and placed 267' from the blast. The pressure transducer was positioned in front of the tent.

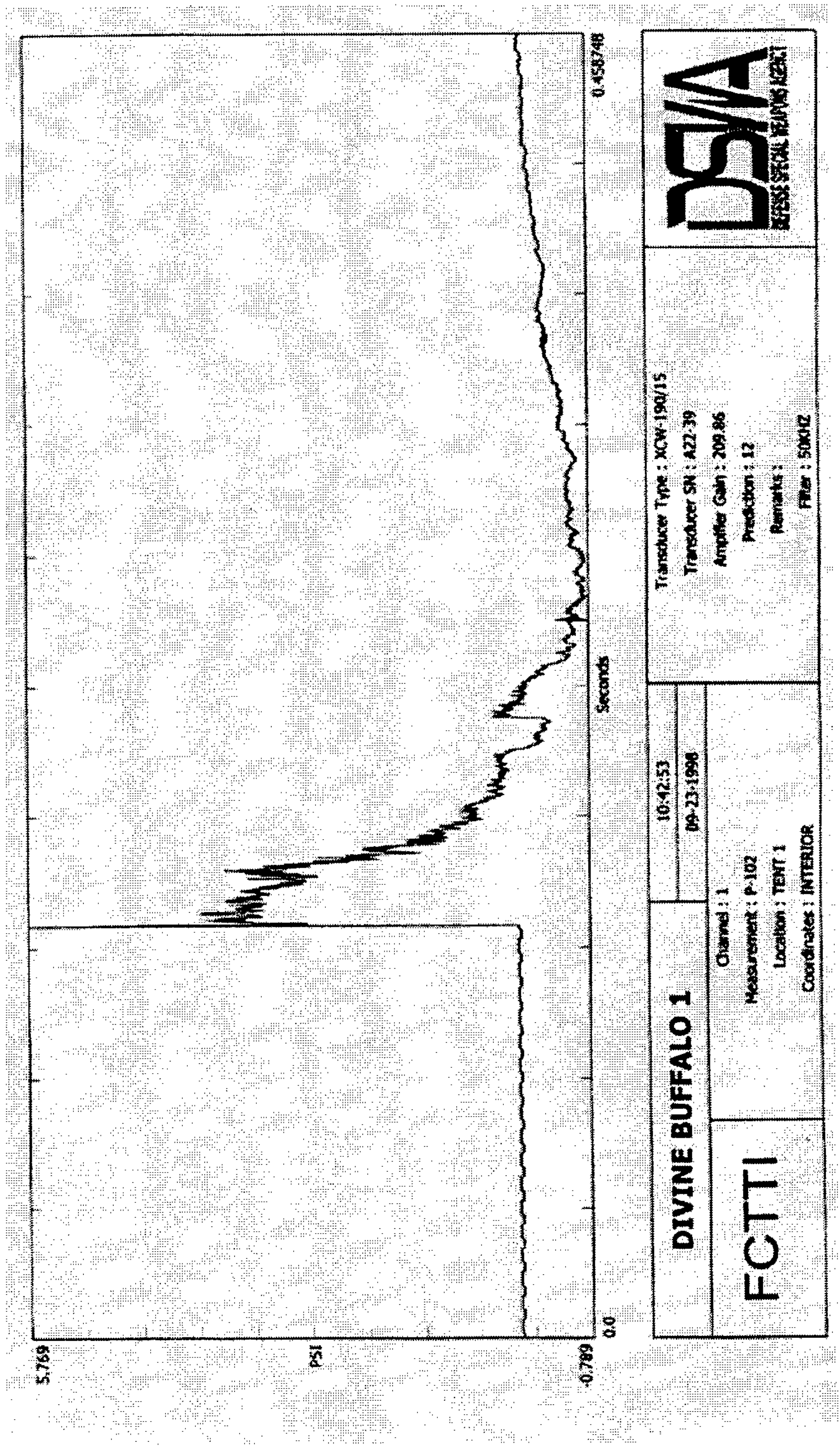


Figure 6.7: Tent 1 was unprotected and placed 267' from the blast. The pressure transducer was positioned inside the tent at 272' from the blast.

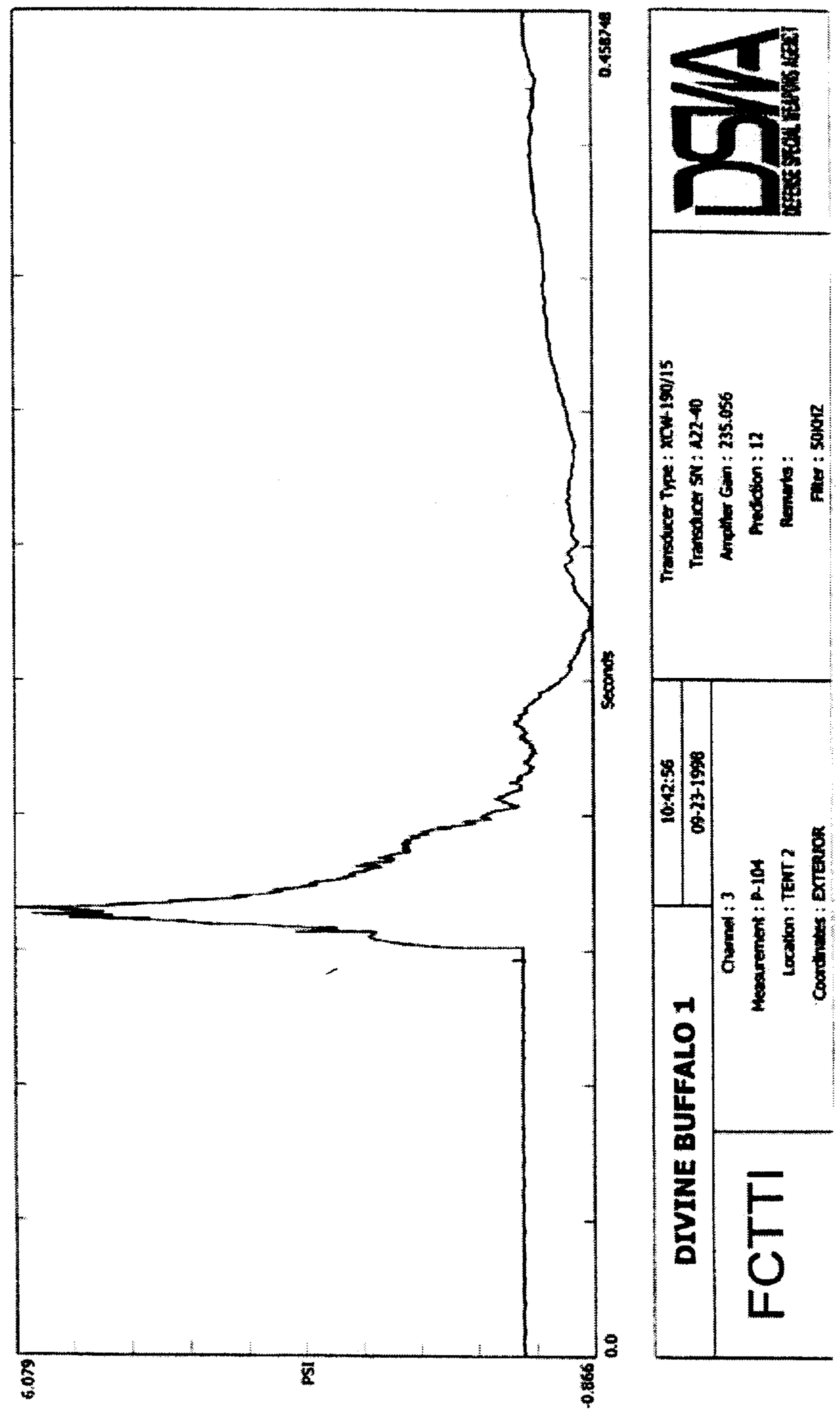


Figure 6.8: Tent 2 was located behind the Maccaferri FlexMac Revetment Module Wall. The pressure transducer was located between the wall and the tent at a distance of 270' from the blast.

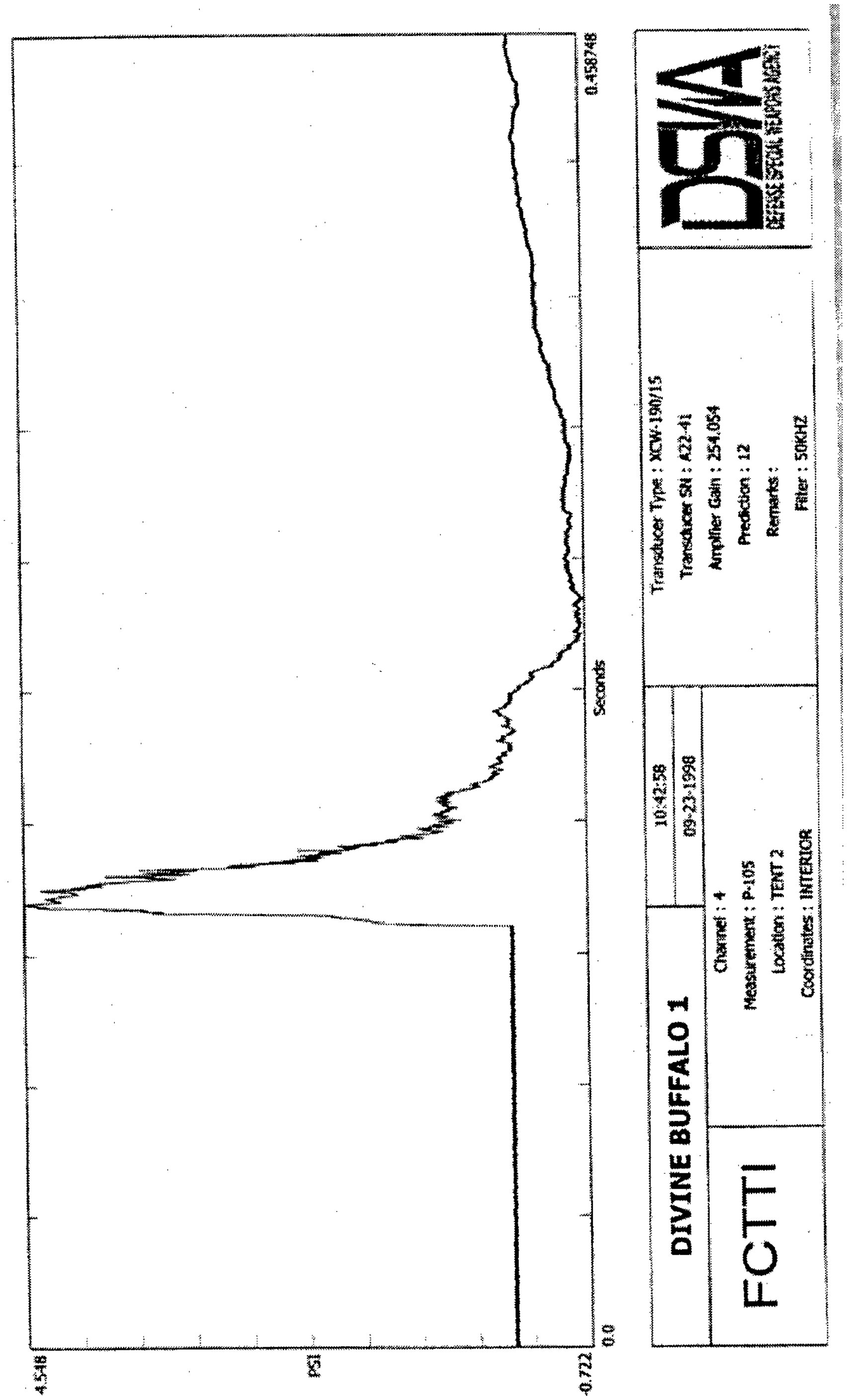


Figure 6.9: Tent 2 was located behind the Maccaferri FlexMac Revetment Module Wall. The pressure transducer was located inside the tent at a distance of 278' from the blast.

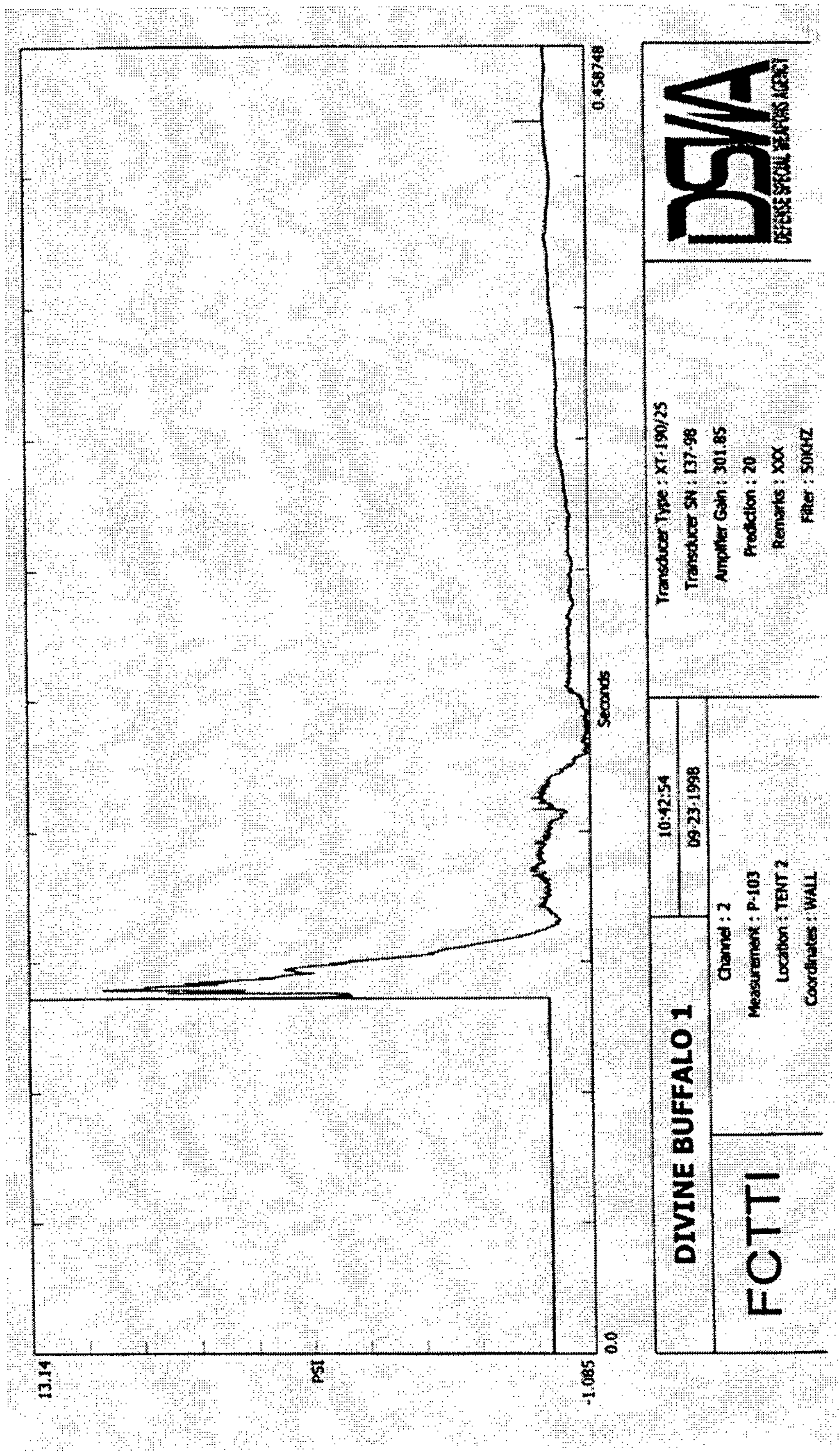


Figure 6.10: The Maccaferri FlexMac Revetment Module Wall was located 267' from the blast. The pressure transducer was located in front of the wall.

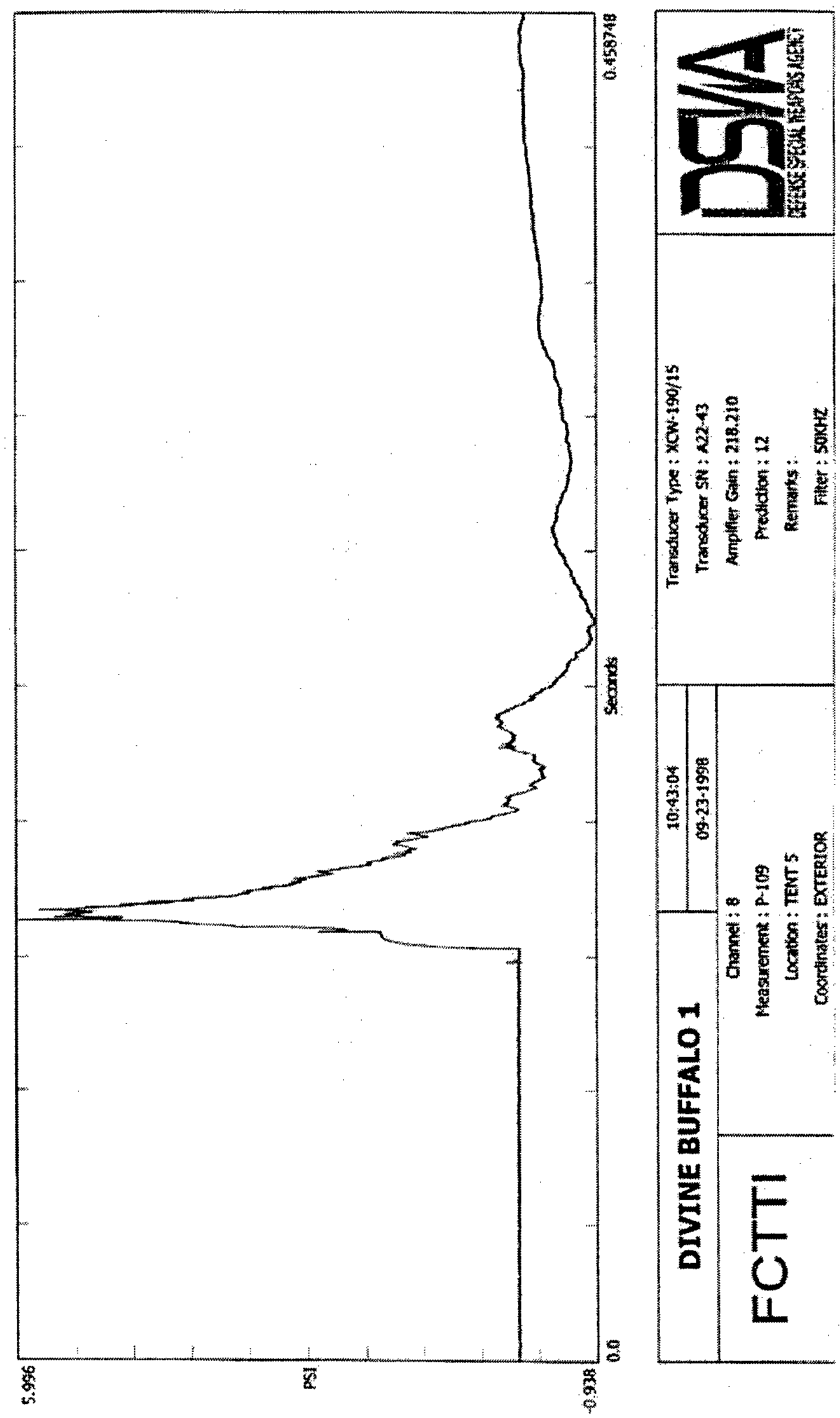


Figure 6.11: Tent 5 was located behind the Hesko-Bastion Concertainer Revetment Module Wall. The pressure transducer was located between the wall and the tent at 270' from the blast.

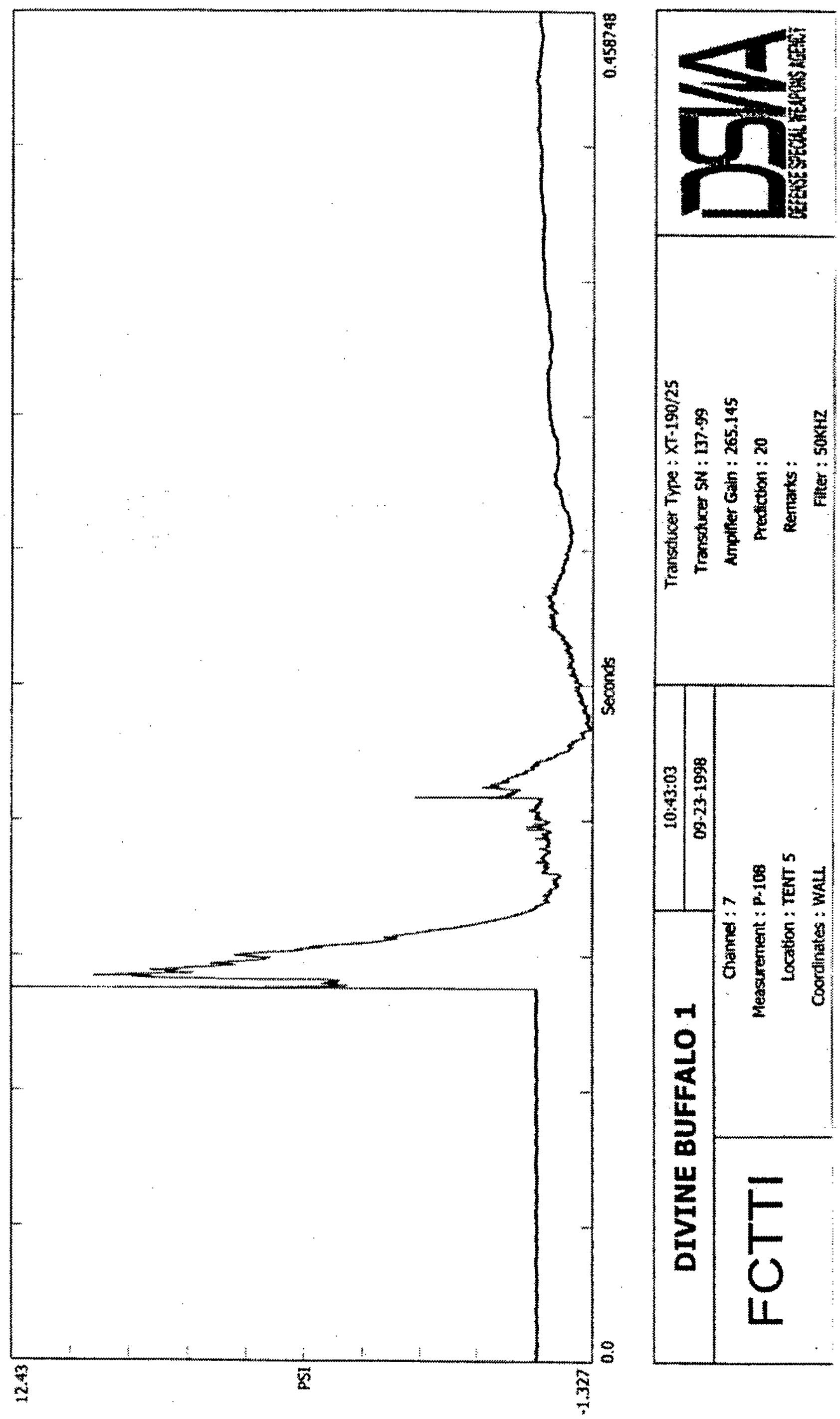


Figure 6.12: The Hesko-Bastion Concertainer Revetment Module Wall was located 267' from the blast. The pressure transducer was located in front of the wall.

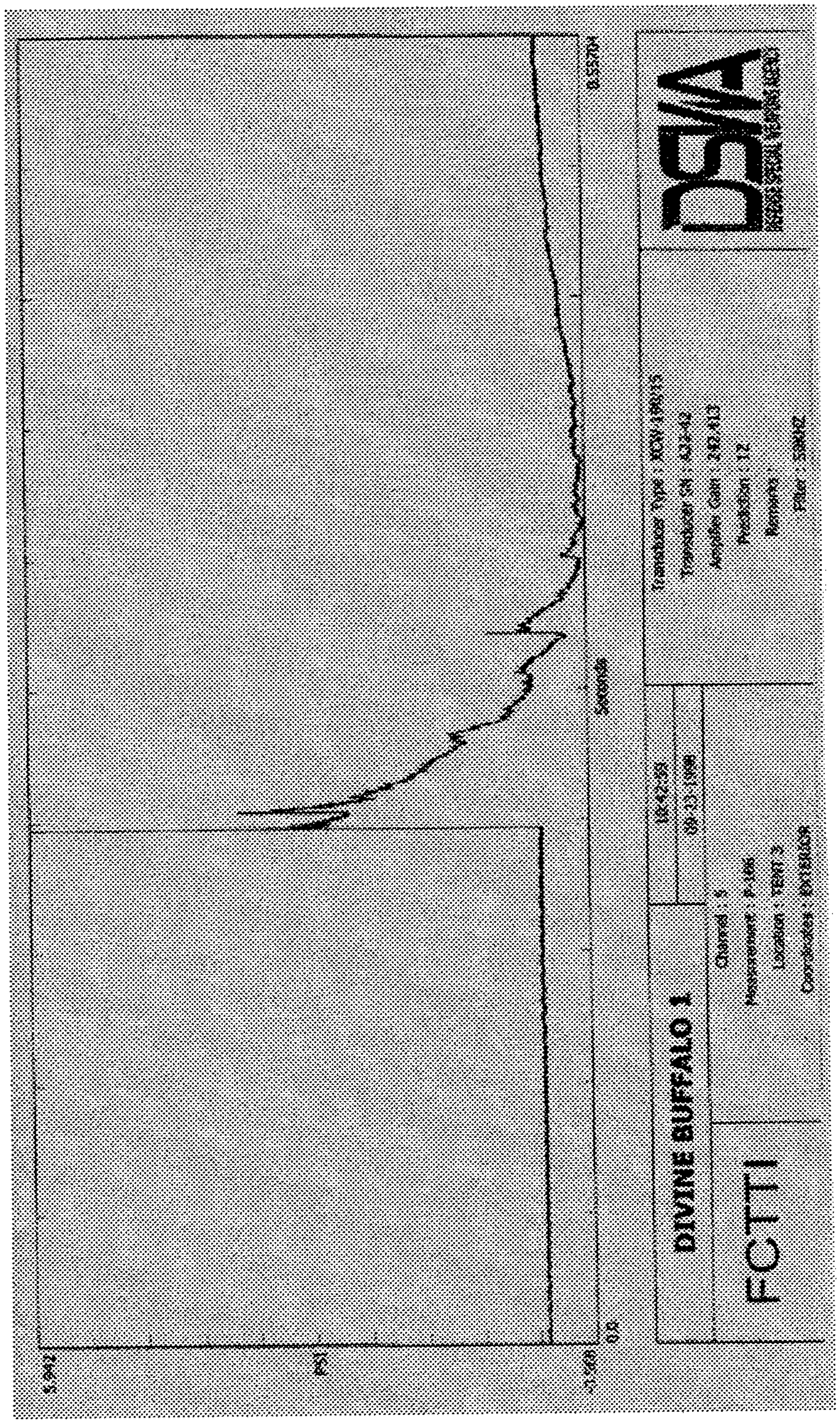


Figure 6.13: Tent 3 was unprotected and placed 370' from the blast. The pressure transducer was placed in front of the tent.

Table 6.1: Measured Pressures on Test Articles

Shelter	Interior Pressure (psi)	Exterior Pressure (psi)	Wall Pressure (psi)	Distance from Blast (ft)
GPS 1	**3.81** (6.2)*	**10.07** (6.1)*	-	277/267
GPS 2	**4.17** (6.4)*	**6.26** (6.3)*	**19.41** (6.5)*	283/270/250
Tent 1	**5.77** (6.7)*	**11.3** (6.6)*	-	272/267
Tent 2	**4.55** (6.9)*	**6.08** (6.8)*	**13.14** (6.10)*	278/270/262
Tent 5	-	**6.00** (6.11)*	**12.43** (6.12)*	270/262
Tent 3	-	**5.94** (6.13)*	-	370

*Denotes the figure for each specific pressure

C. DISCUSSION OF PRESSURE RESULTS

For the rest of the discussion, any reference to pressures will be to the pressures recorded, not calculated pressures. Analyzing the magnitude of the pressures, the results show that the structures that were protected using blast mitigation walls have significantly lower external pressures and comparable internal pressures.

For instance, the external pressures for GPS 2, Tent 2 and Tent 5 are 6.26 psi, 6.08 psi and 6.00 psi respectively. The external pressures for the unprotected structures, GPS 1 and Tent 1, are 10.07 psi and 11.3 psi. Comparing the results, one might deduce that the blast mitigation walls were successful in protecting the structures because they show pressures that are about 4 psi less than the unprotected structures.

However, the important thing to remember is that the external pressure

transducers were located three feet behind the blast mitigation walls and were raised about four feet from ground level. The structures were all located six feet behind the blast mitigation walls and the interior pressure transducers were placed in the middle of the structures.

The more accurate representation of what happened during the test comes from the internal pressures. The protected tent, Tent 2, showed a pressure of 4.55 psi versus the unprotected tent, Tent 1, which had a pressure of 5.77 psi. In this case, the tent was shielded from the blast using a Maccaferri FlexMac Revetment Module Wall. The results show that the wall actually reduced the pressure on the shelter by a small amount. However, the protected general-purpose shelter, GPS 2, actually had a higher internal pressure than the unprotected general-purpose shelter. GPS 2 had a pressure of 4.17 psi versus GPS 1, which had a pressure of 3.81 psi. An important point to note for the interior pressures of GPS 1 and 2 is that the initial peak was not the highest recorded pressure (Figures 6.2/4). The stiff walls forming the GPS's caused the waves to reflect. That is why there are multiple peaks for the interior transducers of the GPS's.

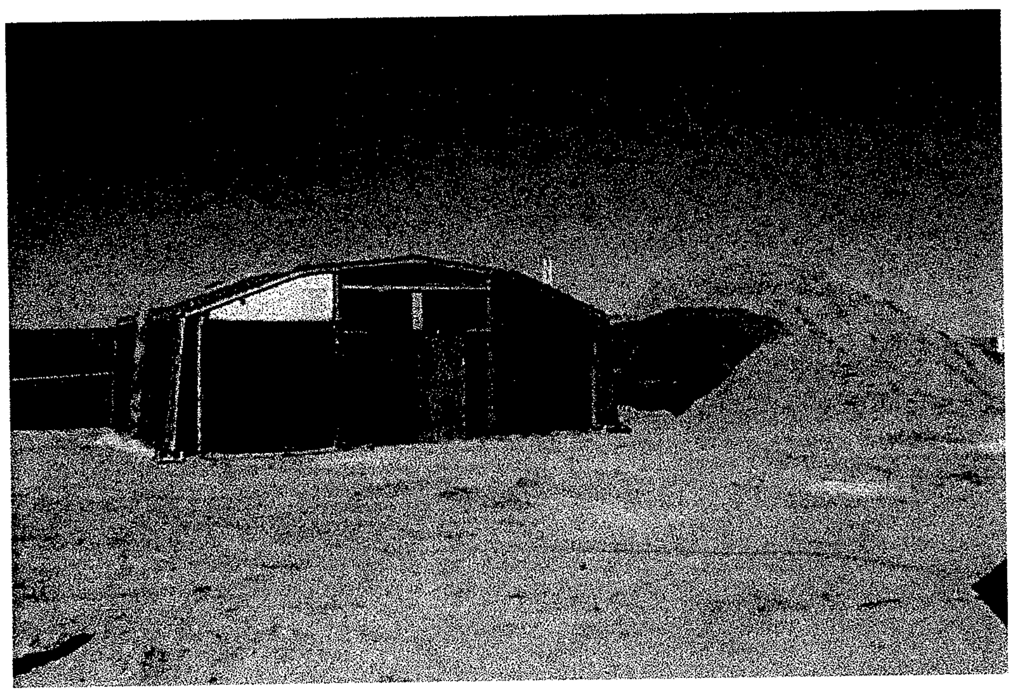
Comparing the wall systems from a pressure standpoint, the modular walls protected the structures against the blast better than the reinforced earth wall. While neither wall system reduced the pressure as much as expected, at least the modular walls decreased the pressure felt by the shelters.

The purpose of using the blast mitigation walls was to reduce the pressures felt by the shelters. The walls were supposed to redirect the blast wave up and beyond the structures that were being protected. Comparing the readings from the external pressure transducers to the internal pressure transducers, it is clear that the "protective bubble" for

the shelters was not nearly large enough. While the external pressures were substantially lower for the shelters that were protected, the walls did not provide any reduction in internal pressures for the tents and general-purpose shelters.

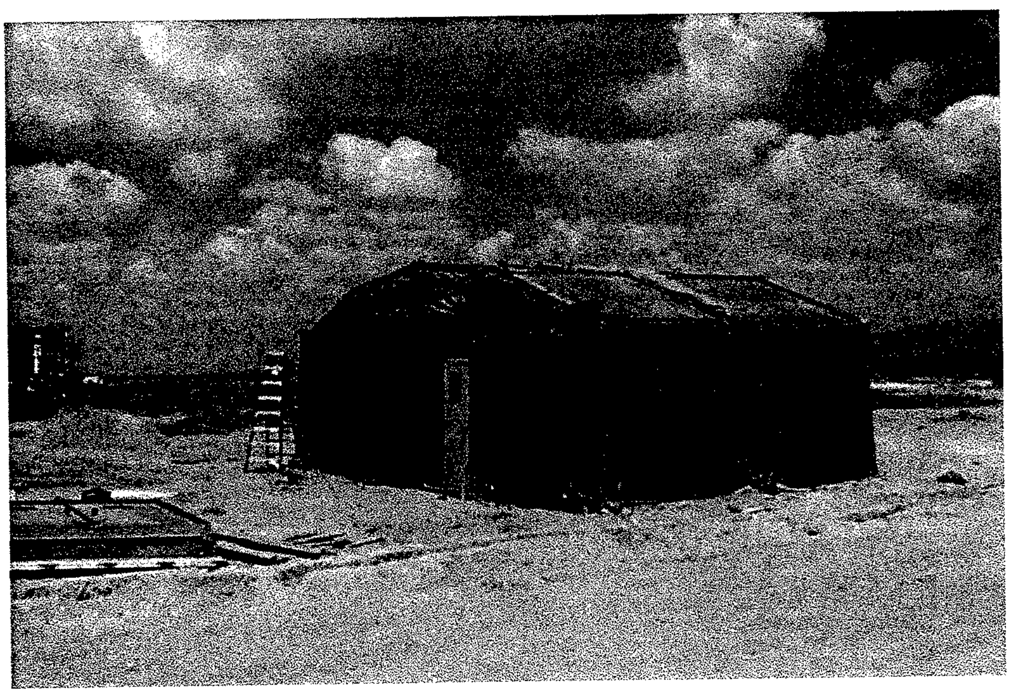
D. DISCUSSION OF PHOTOGRAPHY RESULTS

The results that are seen from the photography only reinforce the results obtained from the pressure transducers. While the information provided from the pressure transducers is extremely valuable, the pictures describe the results in ways words never can. The next series of pictures show before and after shots of the blast mitigation walls, the general-purpose shelters and the tents.

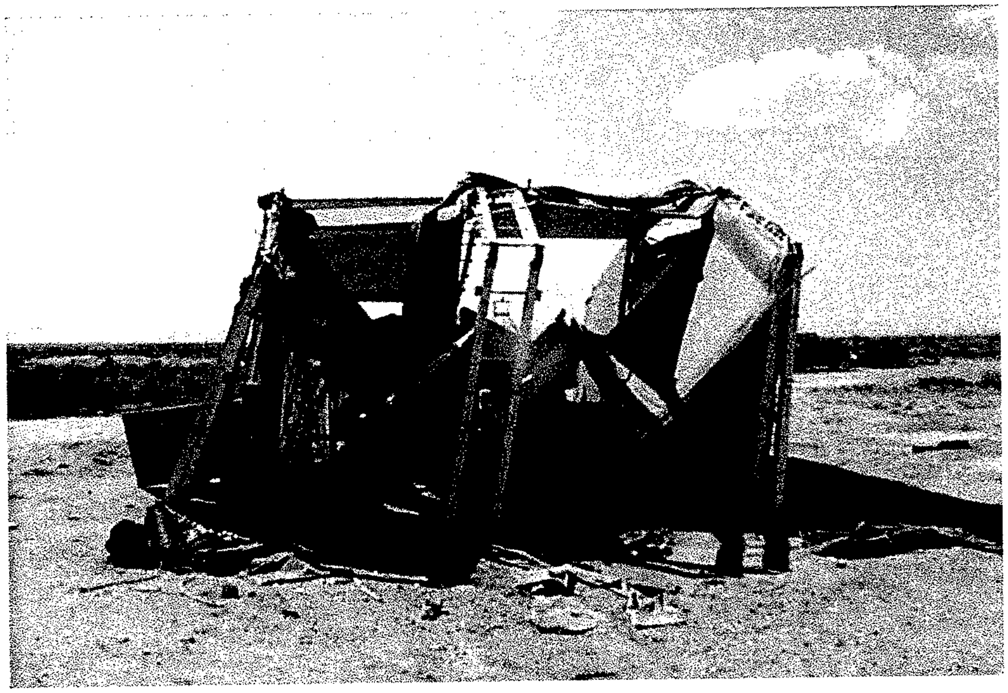


Photograph 6.1: GPS 2 behind the reinforced earth wall before the blast.

Photograph 6.2: GPS 2 after the blast.

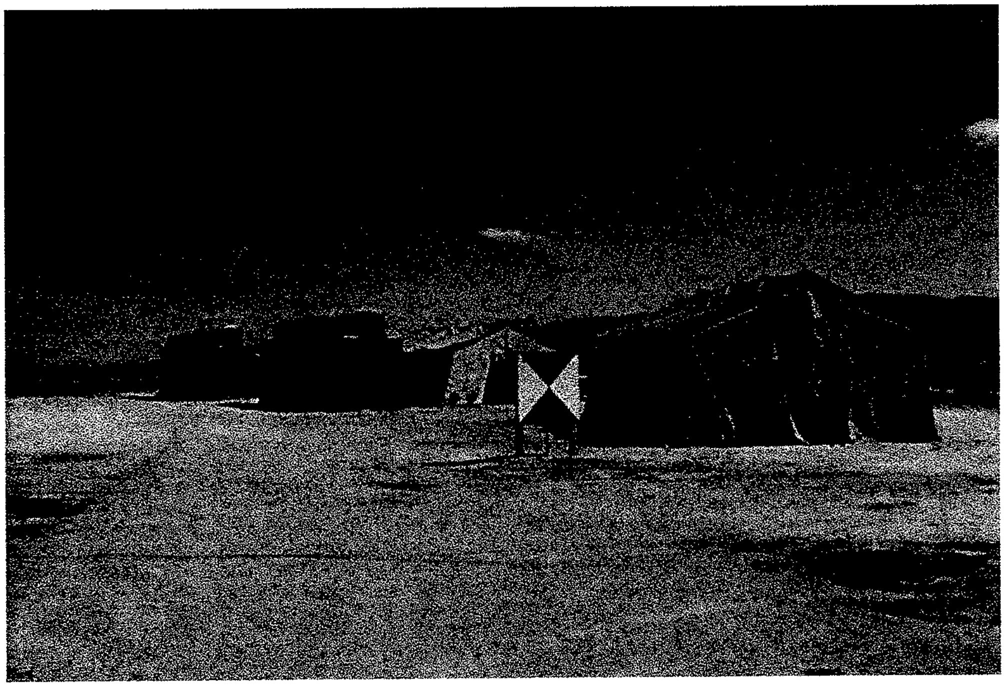


Photograph 6.3: GPS 1 before the blast.

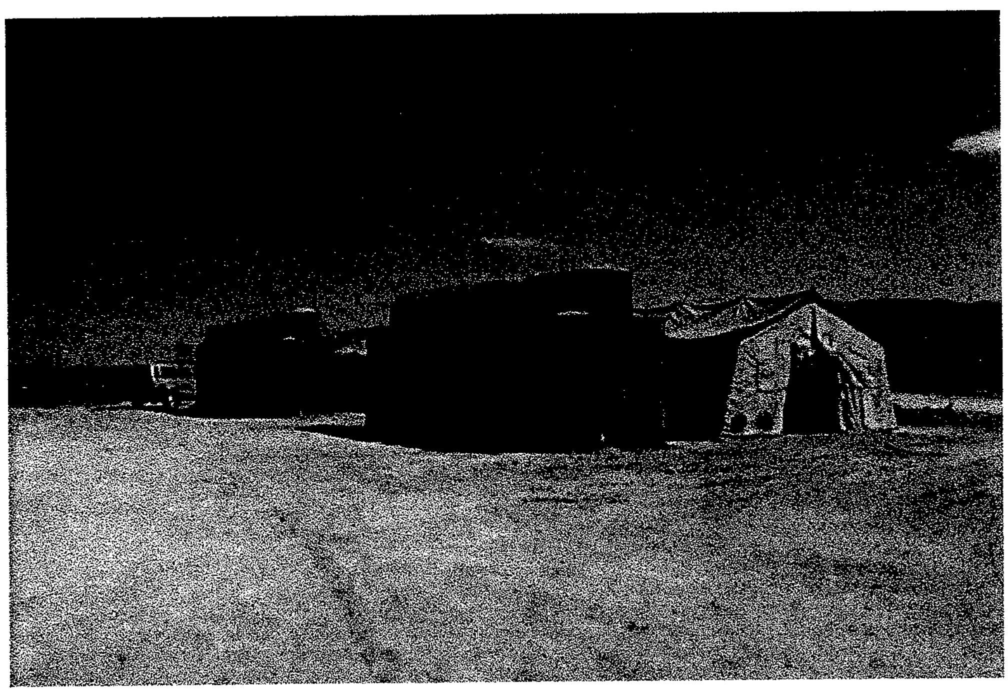


Photograph 6.4: Exterior of GPS 1 after the blast (front view).

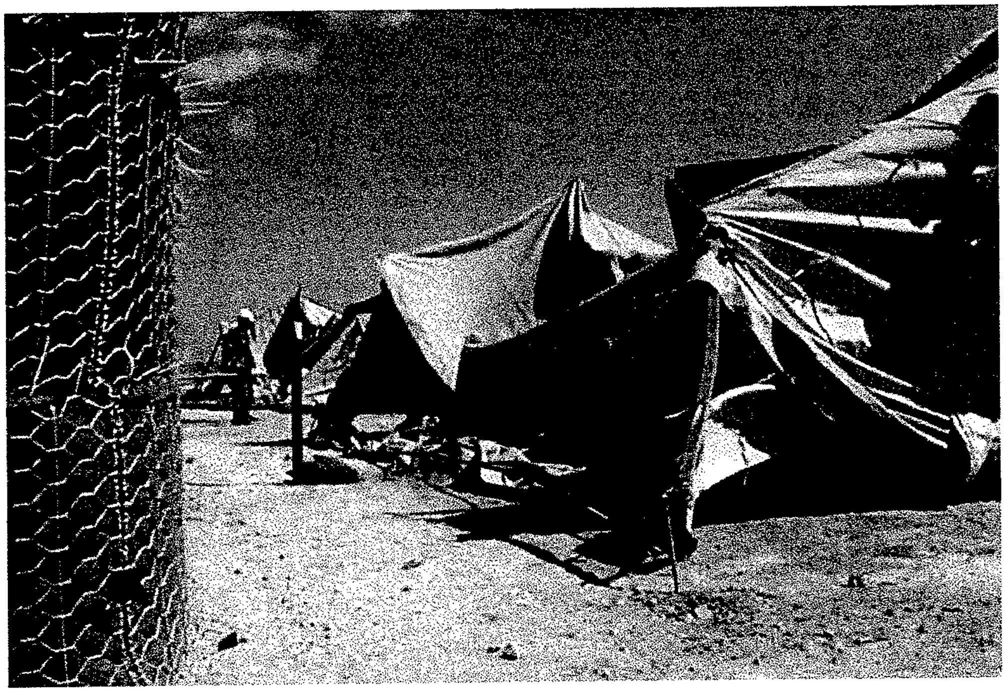
Photograph 6.5: Interior of GPS 1 after the blast.



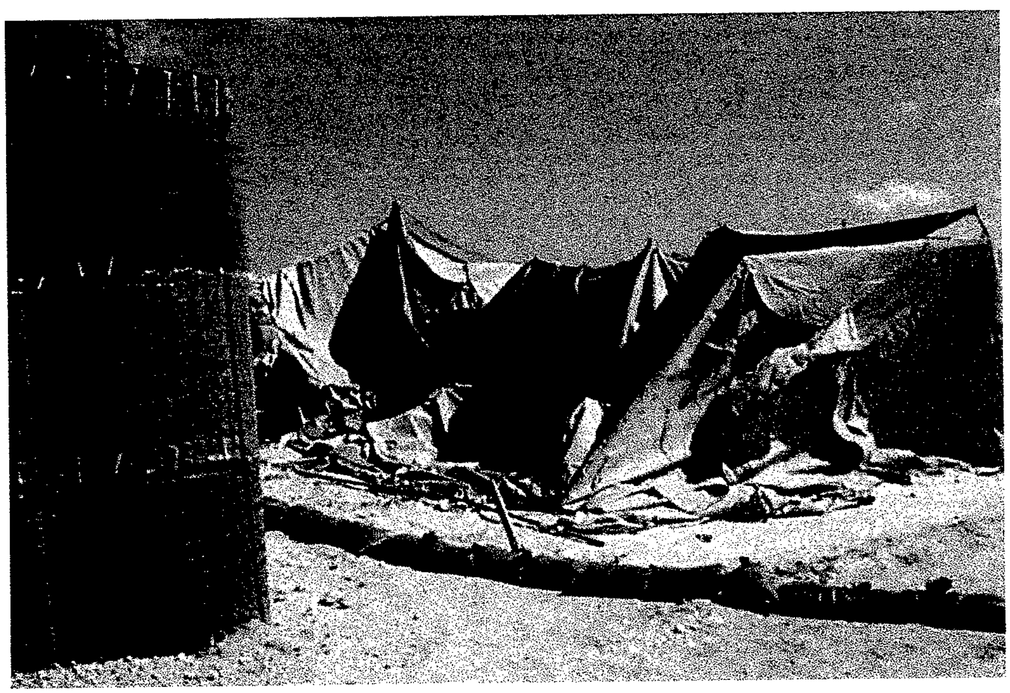
Photograph 6.6: Tents 1, 2 and 5 before the blast. Tents 2 and 5 behind Maccaferri and Hesko-Bastion Revetment Module Walls.



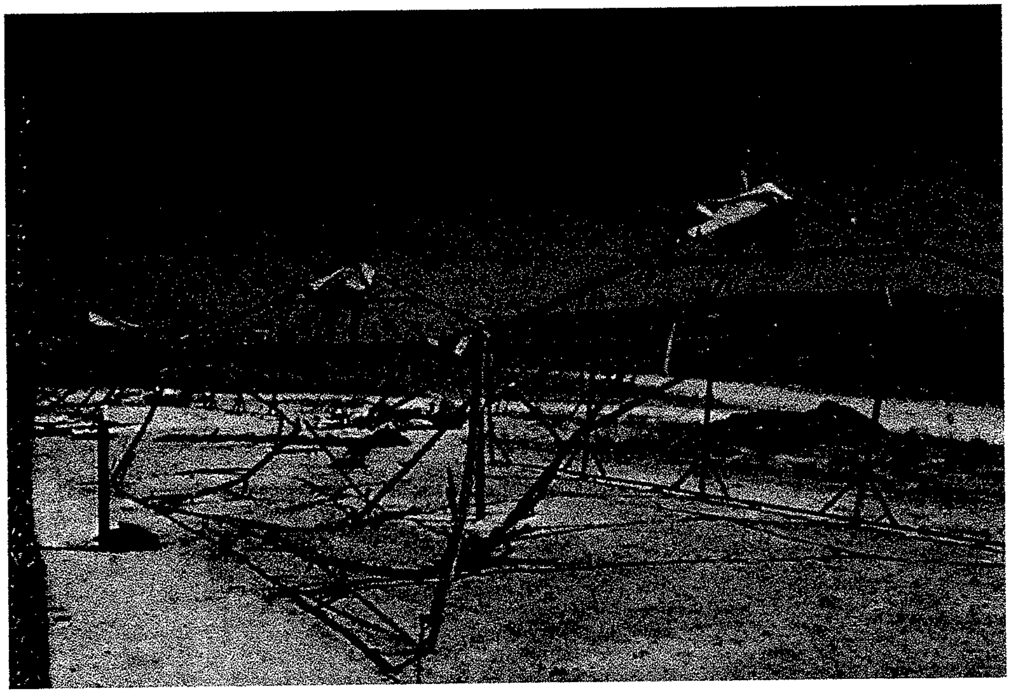
Photograph 6.7: Tents 2 and 5 and modular walls before the blast.



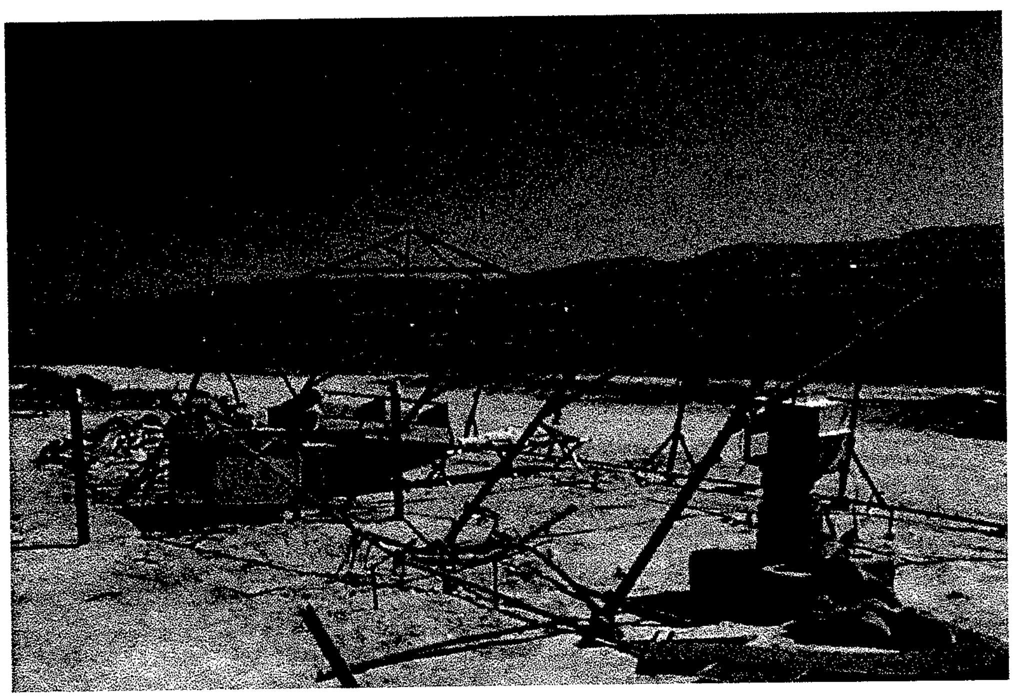
Photograph 6.8: Tent 2 and Maccaferri FlexMac wall after the blast.



Photograph 6.9: Tent 5 and Hesko-Bastion Concertainer wall after the blast.

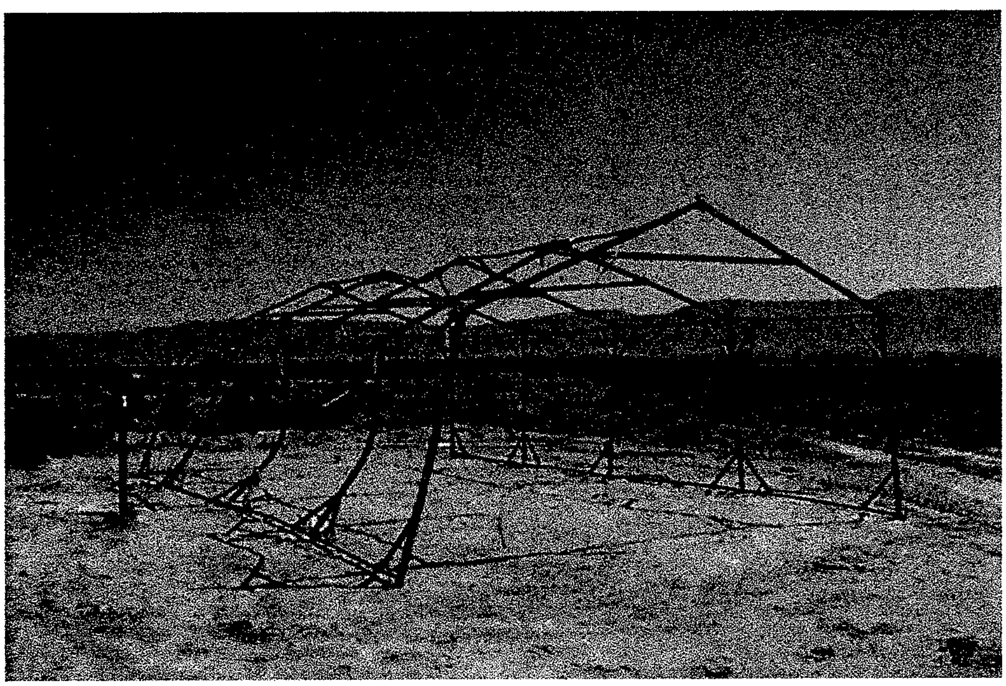


Photograph 6.10: Frames of Tents 2 and 5 after the blast. The tents' skins were removed to show the tents' structural failure.



Photograph 6.11: Frame of Tent 1 after the blast. This tent was unprotected but located 267' from the blast.

Photograph 6.12: Tent 3 before the blast. This tent was located 370' from the blast.



Photograph 6.13: Frame of Tent 3 after the blast. This tent was unprotected but located at 370' from the blast.

The photographs depict what Table 2.3 predicts for butler buildings and tent structures. For expedient facilities, failure could be a subjective criteria. Failure could be considered a certain amount of deflection to engineers, but it could also be determined by whether or not the structure is still standing. With respect to the DIVINE BUFFALO Test, all the facilities were left standing but they cannot be considered functional. For this report, all the facilities failed due to the blast effects.

General Purpose Shelters:

The general-purpose shelter behind the reinforced earth wall (Photograph 6.2) seems to have received most of its damage from the blast pressure that came around the sides of the reinforced earth wall. Although there is not a photograph showing the front of the general-purpose shelter, the front panels of GPS 2 were still attached to the frame. They buckled and deflected into the GPS but they did not come loose and fly through the structure. The side panels, particularly those connecting the side walls to the roof, came apart from the frame. It cannot be determined whether this was a result of the blast pressure directly or due to the blast pressure's total effect on the GPS. By the total effect, it appears that the blast pressure came around the side of the reinforced earth wall and caused the GPS to lean towards one side. When the GPS leaned to one side, the fixity of the side panels on the top could have come loose. Looking at the roof panels of the GPS, there is very acceptable deformation until the rear panels of the GPS are analyzed. The front and middle roof panels showed some dishing but they did not come away from the framework. The roof panels towards the rear of the GPS sustained significant deformation and actually pulled away from the frame. Although this statement is only speculation, had the reinforced earth wall been twice as wide and somewhat taller, the

GPS would have survived the blast. There would still have been deformation, but the GPS would not have been destroyed.

Comparing GPS 1 and GPS 2 shows the same kind of destruction except for the roof panels. Both general-purpose shelters have panels that were forced from the frame. However, the damage to GPS 1 is more pronounced. For instance, both front panels and several roof panels were torn from the frame (Photograph 6.4). In contrast, except for the rear roof panels, the roof panels of GPS 2 weren't separated from the frame. The difference shows that the reinforced earth wall in front of GPS 2 provided some blast mitigation for the front section of the general-purpose shelter. However, both general-purpose shelters are severely tilted due to the blast effects. Additional lateral cross bracing within the general-purpose shelters might eliminate this occurrence.

Tents:

Tents 2 and 5 incurred the same amount and type of damage resulting from the blast. They will be analyzed together and compared to Tent 1, which did not have a blast mitigation wall in front of it. Lastly, Tent 3 will be analyzed to show the benefit of adequate standoff distance.

Tents 2, 5, and 1 all failed as a result of the blast. Photographs 6.10 and 6.11 show the tents with their skins removed after the blast. Tents 2 and 5 received significant damage to the entire frame section closest to the blast while the frame section away from the blast is not separated but the hinges yielded and caused the tents to lean away from the blast. This is most like a result of the blast wave coming over the top and around the sides of the blast mitigation walls. Tent 1 also showed the same types of damage but the magnitude of damage was worse. In addition to the frame breaking and leaning away

from the blast, the frame section nearest the tent also pulled away from the stakes and the entire section moved approximately 14" away from the blast. Photograph 6.11 shows the bottom frame section of Tent 1 on top of a table that was situated inside. This contrast between Tents 2/5 and 1 shows that the blast mitigation walls provided some protection to Tents 2 and 5 even though they were ultimately destroyed. This can also be observed from Table 6.1 that shows external reflective pressures around 6 psi for the tents that were protected by blast mitigation walls as compared to the unprotected tent which has an external reflective pressure around 11 psi. The tents behind blast mitigation walls failed for the same reason as GPS 2 behind the reinforced earth wall. The blast mitigation walls weren't long enough to prevent the blast wave from coming around the walls and destroying the facilities.

Tent 3 (Photograph 6.13) shows interesting results because it reinforces the concept of standoff distance. Although the tent was only located 100' further from the blast than GPS 1 and 2 and Tents 2, 5, and 1, it sustained minimal effects from the blast. In fact, it is still functional even though it did not have a blast mitigation wall in front of it. The only damage to the tent's frame is minimal bending of the members closest to the blast.

The following table compares the material cost, the equipment required to construct the walls, the time required for construction and the deformation of the walls as a result of the blast. The Maccaferri FlexMax requires the same equipment and amount of time for construction. The material cost was not available so it is not included in the table.

Table 6.2: Comparison of Construction Elements for Blast Mitigation Walls

Wall Type	Material Cost ($)	Construction Equipment Required	Time Required (days)	Deformation From Blast
Hesko-Bastion Concertainer Revetment Wall	4300	Loader Excavator	1	None
Reinforced Earth Wall	4500	Loader Backhoe	4	None

This table compare the design cost for the two different systems. The cost is somewhat deceptive since it considers a thirty-six foot wide reinforced earth wall that was supposed to have three 80° faces. The same wall with one 80° rear face would cost $1,500 dollars for the reinforcement and an additional $500 dollars for the wire fabric baskets. However, the real benefit using the Hesko-Bastion Concertainer Revetment Walls is the ease of construction and the quickness in which the walls can be erected. For short-notice construction, the modular walls would be a better option than constructing reinforced earth walls.

SECTION VII

CONCLUSIONS

1. When adequate standoff distance cannot be obtained, current hardening methods using expedient construction are inadequate. Pressures exceeded 3 psi.
2. Obtaining adequate standoff distance is the best method to mitigate blast effects.
3. The Hesko-Bastion Concertainer Revetment Wall and the Maccaferri FlexMac Revetment Wall are easier and faster to construct than reinforced earth walls.
4. The facilities were destroyed due to blast effects, not fragmentation.
5. The blast mitigation walls showed no deformation as a result of the blast.
6. Blast mitigation walls must be considerably wider and taller than the facilities they are intended to protect. Future testing is required to determine a suitable ratio.

SECTION VIII

FUTURE TESTING

1. This test should be performed again with the following changes:

    a. The reinforced earth wall needs to be constructed using wire baskets.

    b. Several different width:height ratios should be used for the blast mitigation walls to determine which ratio, if any, can mitigate the blast effects.

    c. More pressure transducers are required inside the facilities to determine where the blast wave converges on the facility being protected.

    d. Following the initial test, a separate test needs to be performed on the blast mitigation walls. The purpose is to determine the resistance of the walls to direct hits (simulating artillery shells). This test would compare the survivability of the different blast mitigation walls.

    e. To more accurately predict the performance of the blast mitigation walls, all facilities should be the same, either tents or general-purpose shelters.

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APPENDIX A

```
*************************************************************
*****                    R S S                          *****
*****           Reinforced Slope Stability              *****
*****    (c)1992-1996 by GEOCOMP Corp, Concord, MA
*****   licensed to FHWA for distribution by FHWA only  *****
*************************************************************
```

File :
Date : Mon 09-21-98, 14:10:08
Name : Ron Pieri
Problem Title : Reinforced Earth Wall
Description : 14' High with Leeward Face approx 80 degrees
Remarks :

```
*************************************************************
*****                 INPUT DATA                        *****
*************************************************************
```

Data for Generating Simple Problem

Note: The following data reflect the data used by Simple Problem to automatically generate a data file. Changes made by editing that data are not reflected in the Simple Problem data.

X-Coordinate for Toe of Slope : 100.00 ft
Y-Coordinate for Toe of Slope : 100.00 ft
Height of Slope : 14.00 ft
Angle of Slope : 80.0 deg
Angle Above Crest of Slope : 0.0 deg
Surcharge Above Crest of Slope : 0.0 psf
Depth to Water from Crest of Slope : 100.00 ft
Unit Weight of Soil in Slope : 110.00 pcf
Cohesion for Soil in Slope : 0.00 psf
Friction Angle for Soil in Slope : 32.0 deg

Unit Weight of Soil in Foundation : 110.00 pcf
Cohesion for Soil in Foundation : 0.00 psf
Friction Angle for Soil in Foundation : 32.0 deg
Required Internal Factor of Safety : 1.50
Required Sliding Factor of Safety : 1.50

Profile Boundaries

Number of Boundaries : 4
Number of Top Boundaries : 3

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	86.00	100.00	100.00	100.00	2
2	100.00	100.00	102.47	114.00	1
3	102.47	114.00	144.47	114.00	1
4	100.00	100.00	144.47	100.00	2

Soil Parameters

Number of Soil Types : 2

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	110.0	110.0	0.0	32.0	0.00	0.0	0
2	110.0	110.0	0.0	32.0	0.00	0.0	1

Piezometric Surfaces

Number of Surfaces : 1
Unit Weight of Water : 62.43 pcf

Piezometric Surface No. : 1
Number of Coordinate Points : 2

Point No.	X (ft)	Y (ft)
1	86.00	14.00
2	144.47	14.00

****

***** TRIAL SURFACE GENERATION *****

****

Data for Generating Circular Surfaces

Number of Initiation Points : 10
Number of Surfaces From Each Point : 10
Left Initiation Point : 100.00 ft
Right Initiation Point : 101.85 ft
Left Termination Point : 102.47 ft
Right Termination Point : 131.89 ft
Minimum Elevation : 0.00 ft
Segment Length : 1.62 ft
Positive Angle Limit : 72.00 deg
Negative Angle Limit : 0.00 deg

****
*****   TRIAL SURFACE GENERATION   *****
****

Data for Generating Rankine Block Surfaces

Number of Trial Surfaces : 100
Number of Boxes : 2
Segment Length : 14.00 ft

Box No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Height
1	100.41	100.00	100.82	100.00	0.00
2	101.23	100.00	136.71	100.00	0.00

Data for Reinforcement Analysis

Lowest Elevation for Reinforcement : 101.50 ft
Highest Elevation for Reinforcement : 110.50 ft
Minimum Embedment Length : 4.00 ft
Strength Option : Long Term Strength
Extension Factor : 1.00
Reduction Factor : 7.00
Pullout Factor of Safety : 2.00
Pullout Resistance Factor : 0.60
Embedded Scale Factor : 0.67
Slope Coefficient of Friction : 0.37
Foundation Coefficient of Friction : 0.37

Layer No	Elevation (ft)	Length (ft)	Long Term Strength (lb/ft)	Ultimate Strength (lb/ft)
1	101.50	12.00	2000.00	2000.00
2	104.50	12.00	2000.00	2000.00
3	107.50	12.00	2000.00	2000.00
4	110.50	12.00	2000.00	2000.00

****

***** RESULTS *****

****

Factor of safety calculation has gone through ten iterations

The trial failure surface in question is defined
by the following 11 coordinate points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.00	100.00
2	100.69	101.47
3	101.37	102.93
4	102.06	104.40
5	102.74	105.87
6	103.41	107.35
7	104.08	108.82
8	104.75	110.30
9	105.41	111.78
10	106.07	113.25
11	106.40	114.00

Factor of safety for the preceding specified surface is 0.326

Factor of safety calculation has gone through ten iterations

The trial failure surface in question is defined
by the following 11 coordinate points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.00	100.00
2	100.75	101.43
3	101.50	102.87
4	102.25	104.31
5	102.98	105.75
6	103.71	107.20
7	104.44	108.65
8	105.16	110.10
9	105.87	111.55
10	106.58	113.01
11	107.05	114.00

Factor of safety for the preceding specified surface is 0.344

Factor of safety calculation has gone through ten iterations

The trial failure surface in question is defined
by the following 8 coordinate points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.62	103.50
2	101.31	104.96
3	101.92	106.46
4	102.44	107.99
5	102.88	109.55
6	103.22	111.14
7	103.47	112.74
8	103.60	114.00

Factor of safety for the preceding specified surface is 0.308

Factor of safety calculation has gone through ten iterations

The trial failure surface in question is defined
by the following 8 coordinate points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.82	104.66
2	101.42	106.17
3	101.99	107.68
4	102.54	109.21
5	103.05	110.74
6	103.54	112.29
7	104.01	113.84
8	104.05	114.00

Factor of safety for the preceding specified surface is 0.300

Factor of safety calculation has gone through ten iterations

The trial failure surface in question is defined
by the following 8 coordinate points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	100.82	104.66
2	101.47	106.14
3	102.10	107.64
4	102.71	109.14
5	103.30	110.65
6	103.87	112.16
7	104.42	113.69
8	104.52	114.00

Factor of safety for the preceding specified surface is 0.309

Factor of safety calculation has gone through ten iterations

The trial failure surface in question is defined
by the following 7 coordinate points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	101.03	105.83
2	101.63	107.33
3	102.21	108.84
4	102.77	110.36
5	103.32	111.89
6	103.85	113.42
7	104.04	114.00

Factor of safety for the preceding specified surface is 0.302

Factor of safety calculation has gone through ten iterations

The trial failure surface in question is defined
by the following 5 coordinate points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	101.44	108.16
2	102.29	109.54
3	102.90	111.04
4	103.25	112.62
5	103.33	114.00

Factor of safety for the preceding specified surface is 0.339

Factor of safety calculation has gone through ten iterations

The trial failure surface in question is defined
by the following 5 coordinate points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	101.44	108.16
2	101.95	109.69
3	102.45	111.23
4	102.94	112.78
5	103.32	114.00

Factor of safety for the preceding specified surface is 0.297

Factor of safety calculation has gone through ten iterations

The trial failure surface in question is defined
by the following 5 coordinate points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	101.64	109.32
2	102.34	110.78
3	102.93	112.29
4	103.40	113.84
5	103.43	114.00

Factor of safety for the preceding specified surface is 0.313

Factor of safety calculation has gone through ten iterations

The trial failure surface in question is defined
by the following 5 coordinate points

Point No.	X-Surf (ft)	Y-Surf (ft)
1	101.64	109.32
2	102.25	110.82
3	102.79	112.35
4	103.26	113.90
5	103.28	114.00

Factor of safety for the preceeding specified surface is 0.301

Surface No. : 1
Factor of Safety : 1.528
Circle Center X : -171.39 ft
Circle Center Y : 425.59 ft
Circle Radius : 423.87 ft

	Soil	X (ft)	Y (ft)	Width (ft)	Weight (lb)	Load (lb)	Water (lb)	Normal (lb)	Shear (lb)
1	1	100.62	100.52	1.24	410.1	0.0	0.0	398.4	163.0
2	1	101.86	101.56	1.23	1210.4	0.0	0.0	1177.3	481.5
3	1	102.48	102.08	0.01	14.2	0.0	0.0	13.8	5.6
4	1	103.10	102.61	1.23	1546.8	0.0	0.0	1506.4	616.1
5	1	104.33	103.66	1.23	1399.4	0.0	0.0	1364.6	558.1
6	1	105.56	104.72	1.23	1252.4	0.0	0.0	1222.7	500.1
7	1	106.78	105.78	1.22	1105.6	0.0	0.0	1080.8	442.0
8	1	108.00	106.84	1.22	959.1	0.0	0.0	938.9	384.0
9	1	109.22	107.91	1.21	813.0	0.0	0.0	796.9	325.9
10	1	110.43	108.99	1.21	667.2	0.0	0.0	654.8	267.8
11	1	111.64	110.07	1.21	521.7	0.0	0.0	512.7	209.7
12	1	112.84	111.15	1.20	376.6	0.0	0.0	370.6	151.6
13	1	114.04	112.24	1.20	231.9	0.0	0.0	228.5	93.5
14	1	115.24	113.33	1.19	87.6	0.0	0.0	86.4	35.3
15	1	115.90	113.94	0.13	0.8	0.0	0.0	0.8	0.3

Resultant Forces

	X	Y	R	Angle
Weight, lb :	0.00	-10596.91	10596.91	-90.00
Earthquake Load, lb :	0.00	0.00	0.00	0.00
Surcharge Load, lb :	0.00	0.00	0.00	0.00
Top Water, lb :	0.00	0.00	0.00	0.00
Bottom Water, lb :	0.00	0.00	0.00	0.00
Normal Force, lb :	-6780.70	7823.70	10353.17	130.92
Shear Force, lb :	3199.78	2773.21	4234.30	40.92

Surface No. : 2
Factor of Safety : 1.567
Circle Center X : 71.06 ft
Circle Center Y : 139.63 ft
Circle Radius : 49.08 ft

	Soil	X (ft)	Y (ft)	Width (ft)	Weight (lb)	Load (lb)	Water (lb)	Normal (lb)	Shear (lb)
1	1	100.65	100.49	1.29	451.2	0.0	0.0	434.6	173.3
2	1	101.88	101.45	1.18	1193.0	0.0	0.0	1160.3	462.6
3	1	102.51	101.96	0.08	108.2	0.0	0.0	105.2	42.0
4	1	103.16	102.53	1.23	1546.2	0.0	0.0	1520.2	606.2
5	1	104.37	103.61	1.19	1359.9	0.0	0.0	1353.3	539.6
6	1	105.54	104.72	1.15	1175.8	0.0	0.0	1185.7	472.8
7	1	106.68	105.88	1.11	995.0	0.0	0.0	1018.1	406.0
8	1	107.77	107.08	1.07	818.6	0.0	0.0	851.1	339.4
9	1	108.82	108.31	1.03	647.8	0.0	0.0	685.3	273.3
10	1	109.84	109.57	0.99	483.7	0.0	0.0	521.4	207.9
11	1	110.81	110.87	0.95	327.4	0.0	0.0	360.1	143.6
12	1	111.74	112.19	0.91	180.0	0.0	0.0	202.3	80.7
13	1	112.55	113.43	0.71	44.5	0.0	0.0	51.2	20.4

Resultant Forces

	X	Y	R	Angle
Weight, lb :	0.00	-9331.51	9331.51	-90.00
Earthquake Load, lb :	0.00	0.00	0.00	0.00
Surcharge Load, lb :	0.00	0.00	0.00	0.00
Top Water, lb :	0.00	0.00	0.00	0.00
Bottom Water, lb :	0.00	0.00	0.00	0.00
Normal Force, lb :	-6623.05	6690.62	9414.30	134.71
Shear Force, lb :	2667.84	2640.89	3753.89	44.71

Surface No. : 3
Factor of Safety : 1.667
Circle Center X : 39.82 ft
Circle Center Y : 180.25 ft
Circle Radius : 98.71 ft

	Soil	X (ft)	Y (ft)	Width (ft)	Weight (lb)	Load (lb)	Water (lb)	Normal (lb)	Shear (lb)
1	1	101.05	102.83	1.27	433.1	0.0	0.0	425.9	159.7
2	1	102.08	103.66	0.79	703.1	0.0	0.0	695.1	260.6
3	1	102.70	104.17	0.47	503.7	0.0	0.0	498.0	186.7
4	1	103.55	104.88	1.24	1240.5	0.0	0.0	1233.5	462.4
5	1	104.78	105.94	1.22	1081.4	0.0	0.0	1081.8	405.6
6	1	105.99	107.02	1.20	923.4	0.0	0.0	929.7	348.5
7	1	107.19	108.11	1.18	766.9	0.0	0.0	777.2	291.4
8	1	108.36	109.23	1.17	612.0	0.0	0.0	624.6	234.1
9	1	109.52	110.36	1.15	459.1	0.0	0.0	471.9	176.9
10	1	110.66	111.51	1.13	308.4	0.0	0.0	319.5	119.8
11	1	111.77	112.69	1.11	160.2	0.0	0.0	167.3	62.7
12	1	112.66	113.64	0.66	26.1	0.0	0.0	27.5	10.3

Resultant Forces

	X	Y	R	Angle
Weight, lb :	0.00	-7217.97	7217.97	-90.00
Earthquake Load, lb :	0.00	0.00	0.00	0.00
Surcharge Load, lb :	0.00	0.00	0.00	0.00
Top Water, lb :	0.00	0.00	0.00	0.00
Bottom Water, lb :	0.00	0.00	0.00	0.00
Normal Force, lb :	-4820.19	5410.92	7246.54	131.70
Shear Force, lb :	2028.51	1807.05	2716.66	41.70

Surface No. : 4
Factor of Safety : 1.669
Circle Center X : 0.43 ft
Circle Center Y : 199.66 ft
Circle Radius : 140.87 ft

	Soil	X (ft)	Y (ft)	Width (ft)	Weight (lb)	Load (lb)	Water (lb)	Normal (lb)	Shear (lb)
1	1	100.57	100.58	1.14	332.6	0.0	0.0	343.1	128.4
2	1	101.70	101.73	1.13	980.6	0.0	0.0	1017.0	380.6
3	1	102.37	102.42	0.20	247.3	0.0	0.0	257.9	96.5
4	1	102.92	103.01	0.91	1097.6	0.0	0.0	1144.7	428.5
5	1	103.93	104.09	1.10	1198.2	0.0	0.0	1256.8	470.4
6	1	105.02	105.29	1.09	1040.4	0.0	0.0	1097.7	410.8
7	1	106.10	106.49	1.07	884.6	0.0	0.0	939.0	351.4
8	1	107.16	107.72	1.06	731.0	0.0	0.0	780.7	292.2
9	1	108.21	108.95	1.04	579.6	0.0	0.0	623.1	233.2
10	1	109.25	110.19	1.03	430.7	0.0	0.0	466.0	174.4
11	1	110.27	111.45	1.01	284.4	0.0	0.0	309.8	115.9
12	1	111.28	112.72	1.00	140.7	0.0	0.0	154.4	57.8
13	1	112.02	113.68	0.49	17.4	0.0	0.0	19.2	7.2

Resultant Forces

	X	Y	R	Angle
Weight, lb :	0.00	-7965.05	7965.05	-90.00
Earthquake Load, lb :	0.00	0.00	0.00	0.00
Surcharge Load, lb :	0.00	0.00	0.00	0.00
Top Water, lb :	0.00	0.00	0.00	0.00
Bottom Water, lb :	0.00	0.00	0.00	0.00
Normal Force, lb :	-6243.43	5628.22	8405.79	137.97
Shear Force, lb :	2106.57	2336.84	3146.18	47.97

Surface No. : 5
Factor of Safety : 1.685
Circle Center X : 82.03 ft
Circle Center Y : 170.57 ft
Circle Radius : 72.82 ft

	Soil	X (ft)	Y (ft)	Width (ft)	Weight (lb)	Load (lb)	Water (lb)	Normal (lb)	Shear (lb)
1	1	100.78	100.21	1.57	728.0	0.0	0.0	685.6	254.2
2	1	102.02	100.55	0.90	1083.4	0.0	0.0	1018.5	377.7
3	1	102.80	100.77	0.65	947.2	0.0	0.0	890.5	330.2
4	1	103.89	101.11	1.55	2190.7	0.0	0.0	2056.6	762.6
5	1	105.43	101.62	1.53	2089.8	0.0	0.0	1960.2	726.8
6	1	106.96	102.15	1.52	1983.5	0.0	0.0	1859.8	689.6
7	1	108.48	102.72	1.51	1872.1	0.0	0.0	1755.5	650.9
8	1	109.98	103.33	1.50	1755.8	0.0	0.0	1647.5	610.9
9	1	111.47	103.97	1.48	1635.1	0.0	0.0	1536.0	569.5
10	1	112.94	104.64	1.47	1510.3	0.0	0.0	1421.0	526.9
11	1	114.40	105.34	1.45	1381.8	0.0	0.0	1302.9	483.1
12	1	115.84	106.08	1.43	1250.1	0.0	0.0	1181.7	438.2
13	1	117.27	106.85	1.42	1115.4	0.0	0.0	1057.7	392.2
14	1	118.68	107.65	1.40	978.3	0.0	0.0	931.0	345.2
15	1	120.07	108.48	1.38	839.1	0.0	0.0	801.9	297.3
16	1	121.44	109.34	1.36	698.4	0.0	0.0	670.5	248.6
17	1	122.79	110.23	1.34	556.5	0.0	0.0	537.1	199.1
18	1	124.13	111.15	1.32	414.0	0.0	0.0	401.8	149.0
19	1	125.44	112.10	1.30	271.3	0.0	0.0	264.9	98.2
20	1	126.73	113.08	1.28	128.9	0.0	0.0	126.7	47.0
21	1	127.62	113.79	0.51	11.9	0.0	0.0	11.7	4.4

Resultant Forces

	X	Y	R	Angle
Weight, lb :	0.00	-23441.47	23441.47	-90.00
Earthquake Load, lb :	0.00	0.00	0.00	0.00
Surcharge Load, lb :	0.00	0.00	0.00	0.00
Top Water, lb :	0.00	0.00	0.00	0.00
Bottom Water, lb :	0.00	0.00	0.00	0.00
Normal Force, lb :	-8753.14	20195.83	22011.11	113.43
Shear Force, lb :	7488.55	3245.63	8161.65	23.43

Surface No. : 6
Factor of Safety : 1.700
Circle Center X : 98.05 ft
Circle Center Y : 122.61 ft
Circle Radius : 20.42 ft

	Soil	X (ft)	Y (ft)	Width (ft)	Weight (lb)	Load (lb)	Water (lb)	Normal (lb)	Shear (lb)
1	1	101.21	102.46	1.60	776.4	0.0	0.0	743.0	273.1
2	1	102.24	102.64	0.46	507.6	0.0	0.0	479.7	176.3
3	1	103.03	102.82	1.12	1373.1	0.0	0.0	1297.7	477.0
4	1	104.36	103.21	1.54	1828.7	0.0	0.0	1717.6	631.3
5	1	105.88	103.77	1.50	1683.4	0.0	0.0	1581.4	581.3
6	1	107.34	104.45	1.44	1514.8	0.0	0.0	1432.3	526.4
7	1	108.76	105.24	1.38	1328.1	0.0	0.0	1272.0	467.5
8	1	110.10	106.15	1.31	1129.0	0.0	0.0	1102.6	405.3
9	1	111.37	107.16	1.23	924.0	0.0	0.0	926.3	340.5
10	1	112.55	108.26	1.14	719.5	0.0	0.0	745.9	274.2
11	1	113.64	109.45	1.04	522.3	0.0	0.0	564.2	207.4
12	1	114.63	110.73	0.94	339.0	0.0	0.0	384.8	141.4
13	1	115.52	112.08	0.84	176.1	0.0	0.0	211.9	77.9
14	1	116.25	113.39	0.61	40.9	0.0	0.0	52.8	19.4

Resultant Forces

	X	Y	R	Angle
Weight, lb :	0.00	-12862.81	12862.81	-90.00
Earthquake Load, lb :	0.00	0.00	0.00	0.00
Surcharge Load, lb :	0.00	0.00	0.00	0.00
Top Water, lb :	0.00	0.00	0.00	0.00
Bottom Water, lb :	0.00	0.00	0.00	0.00
Normal Force, lb :	-5746.25	10750.68	12190.01	118.12
Shear Force, lb :	3951.59	2112.13	4480.64	28.12

Surface No. : 7
Factor of Safety : 1.706
Circle Center X : -636.96 ft
Circle Center Y : 914.90 ft
Circle Radius : 1095.81 ft

	Soil	X (ft)	Y (ft)	Width (ft)	Weight (lb)	Load (lb)	Water (lb)	Normal (lb)	Shear (lb)
1	1	101.42	105.21	1.20	374.8	0.0	0.0	380.3	139.3
2	1	102.24	105.96	0.45	335.4	0.0	0.0	340.5	124.7
3	1	102.84	106.50	0.74	613.9	0.0	0.0	623.2	228.2
4	1	103.81	107.39	1.19	867.6	0.0	0.0	881.3	322.8
5	1	105.00	108.49	1.19	722.7	0.0	0.0	734.6	269.0
6	1	106.20	109.59	1.19	578.0	0.0	0.0	587.8	215.3
7	1	107.39	110.69	1.19	433.4	0.0	0.0	441.0	161.5
8	1	108.57	111.79	1.19	289.0	0.0	0.0	294.2	107.8
9	1	109.76	112.89	1.19	144.7	0.0	0.0	147.4	54.0
10	1	110.65	113.72	0.60	18.3	0.0	0.0	18.7	6.8

Resultant Forces

	X	Y	R	Angle
Weight, lb :	0.00	-4377.85	4377.85	-90.00
Earthquake Load, lb :	0.00	0.00	0.00	0.00
Surcharge Load, lb :	0.00	0.00	0.00	0.00
Top Water, lb :	0.00	0.00	0.00	0.00
Bottom Water, lb :	0.00	0.00	0.00	0.00
Normal Force, lb :	-3011.41	3275.00	4449.07	132.60
Shear Force, lb :	1199.38	1102.85	1629.35	42.60

Surface No. : 8
Factor of Safety : 1.721
Circle Center X : 94.10 ft
Circle Center Y : 125.35 ft
Circle Radius : 21.75 ft

	Soil	X (ft)	Y (ft)	Width (ft)	Weight (lb)	Load (lb)	Water (lb)	Normal (lb)	Shear (lb)
1	1	101.58	104.94	1.52	674.5	0.0	0.0	634.1	230.2
2	1	102.41	105.25	0.13	117.1	0.0	0.0	110.4	40.1
3	1	103.14	105.58	1.35	1248.8	0.0	0.0	1177.4	427.4
4	1	104.53	106.28	1.42	1207.7	0.0	0.0	1148.5	416.9
5	1	105.92	107.10	1.36	1031.2	0.0	0.0	994.7	361.1
6	1	107.24	108.04	1.29	846.6	0.0	0.0	833.3	302.5
7	1	108.50	109.06	1.21	659.4	0.0	0.0	666.3	241.9
8	1	109.67	110.18	1.13	475.4	0.0	0.0	496.2	180.2
9	1	110.75	111.38	1.04	300.2	0.0	0.0	326.0	118.4
10	1	111.75	112.66	0.95	139.8	0.0	0.0	159.1	57.8
11	1	112.43	113.66	0.42	15.8	0.0	0.0	19.0	6.9

Resultant Forces

	X	Y	R	Angle
Weight, lb :	0.00	-6716.47	6716.47	-90.00
Earthquake Load, lb :	0.00	0.00	0.00	0.00
Surcharge Load, lb :	0.00	0.00	0.00	0.00
Top Water, lb :	0.00	0.00	0.00	0.00
Bottom Water, lb :	0.00	0.00	0.00	0.00
Normal Force, lb :	-3537.76	5432.09	6482.54	123.08
Shear Force, lb :	1972.11	1284.38	2353.48	33.08

Surface No. : 9
Factor of Safety : 1.722
Circle Center X : 91.67 ft
Circle Center Y : 134.31 ft
Circle Radius : 34.23 ft

	Soil	X (ft)	Y (ft)	Width (ft)	Weight (lb)	Load (lb)	Water (lb)	Normal (lb)	Shear (lb)
1	1	100.98	101.39	1.56	719.6	0.0	0.0	678.2	246.1
2	1	102.12	101.72	0.71	797.6	0.0	0.0	750.0	272.2
3	1	102.89	101.98	0.83	1097.9	0.0	0.0	1032.4	374.6
4	1	104.06	102.41	1.51	1924.6	0.0	0.0	1809.6	656.6
5	1	105.55	103.04	1.48	1785.7	0.0	0.0	1682.7	610.6
6	1	107.02	103.73	1.45	1636.0	0.0	0.0	1548.5	561.9
7	1	108.45	104.49	1.41	1477.2	0.0	0.0	1407.6	510.8
8	1	109.84	105.32	1.37	1311.3	0.0	0.0	1260.8	457.5
9	1	111.19	106.21	1.33	1140.4	0.0	0.0	1109.0	402.4
10	1	112.50	107.16	1.29	966.6	0.0	0.0	953.0	345.8
11	1	113.76	108.18	1.24	792.2	0.0	0.0	793.8	288.0
12	1	114.97	109.25	1.19	619.4	0.0	0.0	632.4	229.5
13	1	116.13	110.38	1.13	450.5	0.0	0.0	470.0	170.5
14	1	117.23	111.57	1.08	288.0	0.0	0.0	307.7	111.7
15	1	118.28	112.80	1.02	134.0	0.0	0.0	147.1	53.4
16	1	119.00	113.72	0.41	12.9	0.0	0.0	14.6	5.3

Resultant Forces

	X	Y	R	Angle
Weight, lb :	0.00	-15153.92	15153.92	-90.00
Earthquake Load, lb :	0.00	0.00	0.00	0.00
Surcharge Load, lb :	0.00	0.00	0.00	0.00
Top Water, lb :	0.00	0.00	0.00	0.00
Bottom Water, lb :	0.00	0.00	0.00	0.00
Normal Force, lb :	-6991.09	12617.21	14424.61	118.99
Shear Force, lb :	4578.14	2536.71	5233.95	28.99

Surface No. : 10
Factor of Safety : 1.726
Circle Center X : 100.68 ft
Circle Center Y : 117.56 ft
Circle Radius : 17.57 ft

	Soil	X (ft)	Y (ft)	Width (ft)	Weight (lb)	Load (lb)	Water (lb)	Normal (lb)	Shear (lb)
1	1	100.81	100.01	1.62	817.1	0.0	0.0	815.0	295.1
2	1	102.04	100.05	0.85	1078.8	0.0	0.0	1046.3	378.9
3	1	102.85	100.13	0.76	1162.2	0.0	0.0	1127.2	408.2
4	1	104.03	100.33	1.59	2392.0	0.0	0.0	2276.6	824.4
5	1	105.60	100.71	1.56	2273.9	0.0	0.0	2142.3	775.8
6	1	107.13	101.23	1.51	2116.2	0.0	0.0	1990.6	720.8
7	1	108.61	101.90	1.45	1924.8	0.0	0.0	1823.1	660.2
8	1	110.02	102.69	1.37	1706.7	0.0	0.0	1641.9	594.5
9	1	111.35	103.61	1.29	1470.1	0.0	0.0	1449.4	524.8
10	1	112.58	104.66	1.19	1224.0	0.0	0.0	1248.2	452.0
11	1	113.72	105.81	1.08	977.5	0.0	0.0	1041.6	377.2
12	1	114.75	107.06	0.97	740.1	0.0	0.0	833.1	301.7
13	1	115.66	108.40	0.85	521.0	0.0	0.0	627.1	227.1
14	1	116.44	109.82	0.71	328.9	0.0	0.0	429.3	155.5
15	1	117.08	111.30	0.58	171.6	0.0	0.0	247.0	89.4
16	1	117.59	112.84	0.44	55.8	0.0	0.0	90.3	32.7
17	1	117.84	113.81	0.07	1.5	0.0	0.0	2.8	1.0

Resultant Forces

	X	Y	R	Angle
Weight, lb :	0.00	-18961.83	18961.83	-90.00
Earthquake Load, lb :	0.00	0.00	0.00	0.00
Surcharge Load, lb :	0.00	0.00	0.00	0.00
Top Water, lb :	0.00	0.00	0.00	0.00
Bottom Water, lb :	0.00	0.00	0.00	0.00
Normal Force, lb :	-8091.10	16031.95	17957.99	116.78
Shear Force, lb :	5805.36	2929.88	6502.80	26.78

CRITICAL ZONE SEARCH IN BOTTOM

Critical Zone Factor of Safety : 1.451

CRITICAL ZONE SEARCH IN MIDDLE

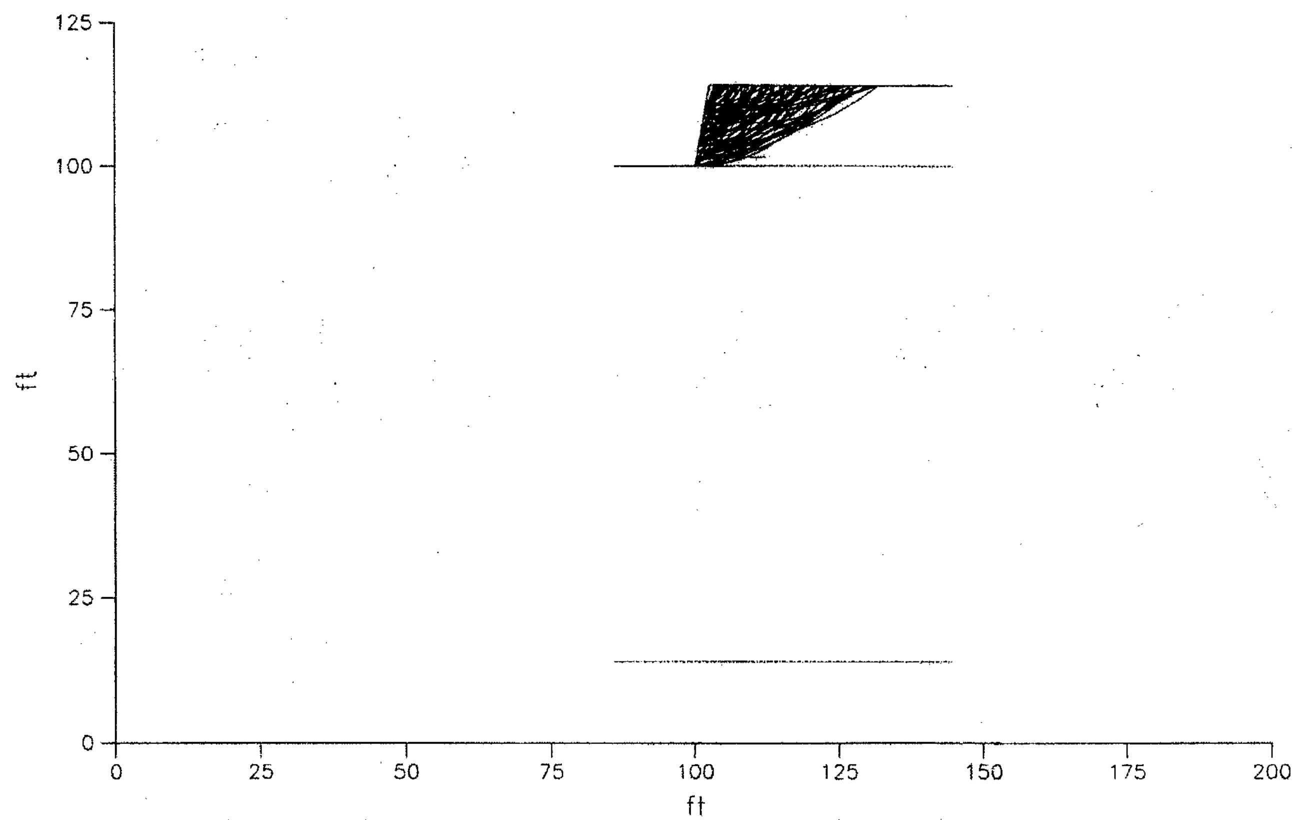
Critical Zone Factor of Safety : 1.702

CRITICAL ZONE SEARCH IN TOP

Critical Zone Factor of Safety : 2.237

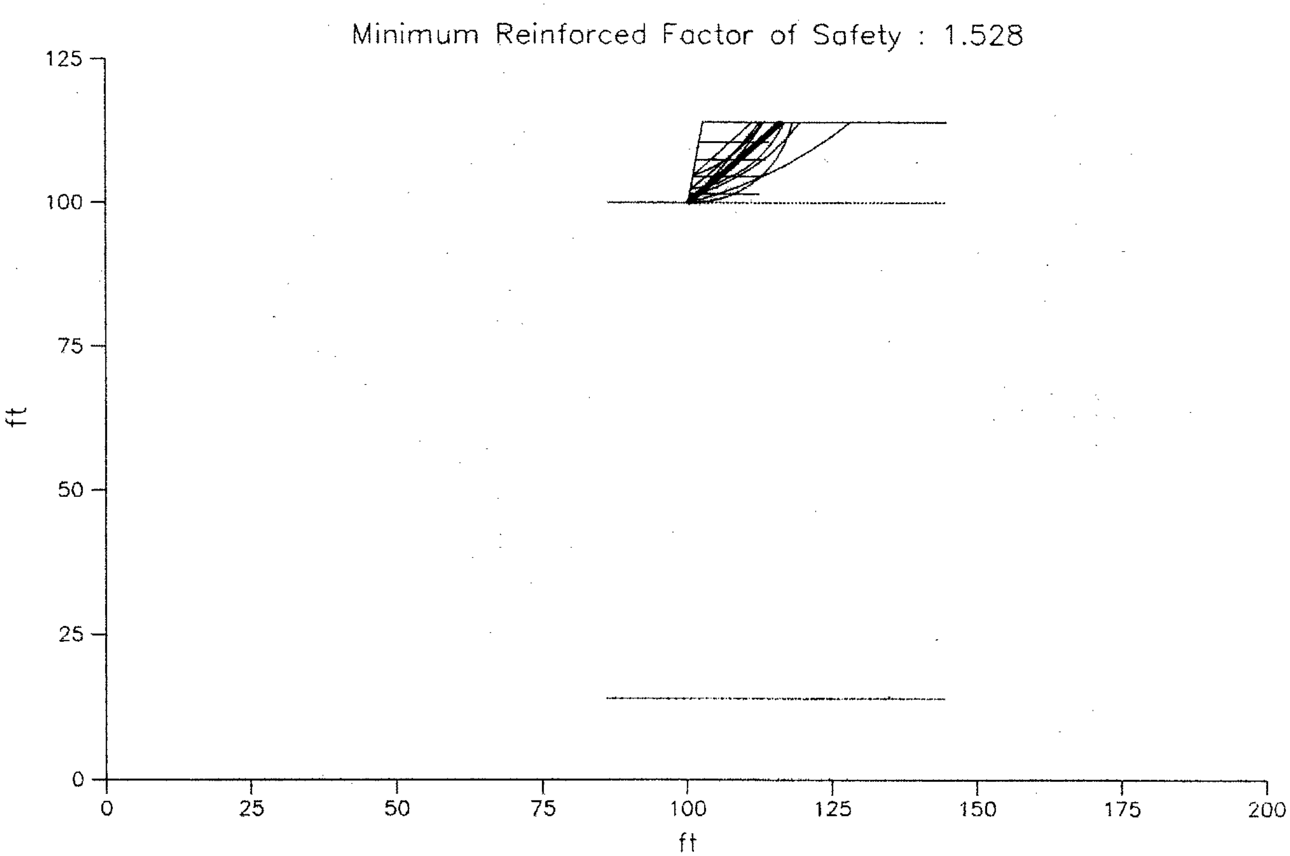
Title : Reinforced Earth Wall
Description : 14' High with Leeward Face approx 80 degrees

Reinforcement Analysis - Search for Critical Surfaces



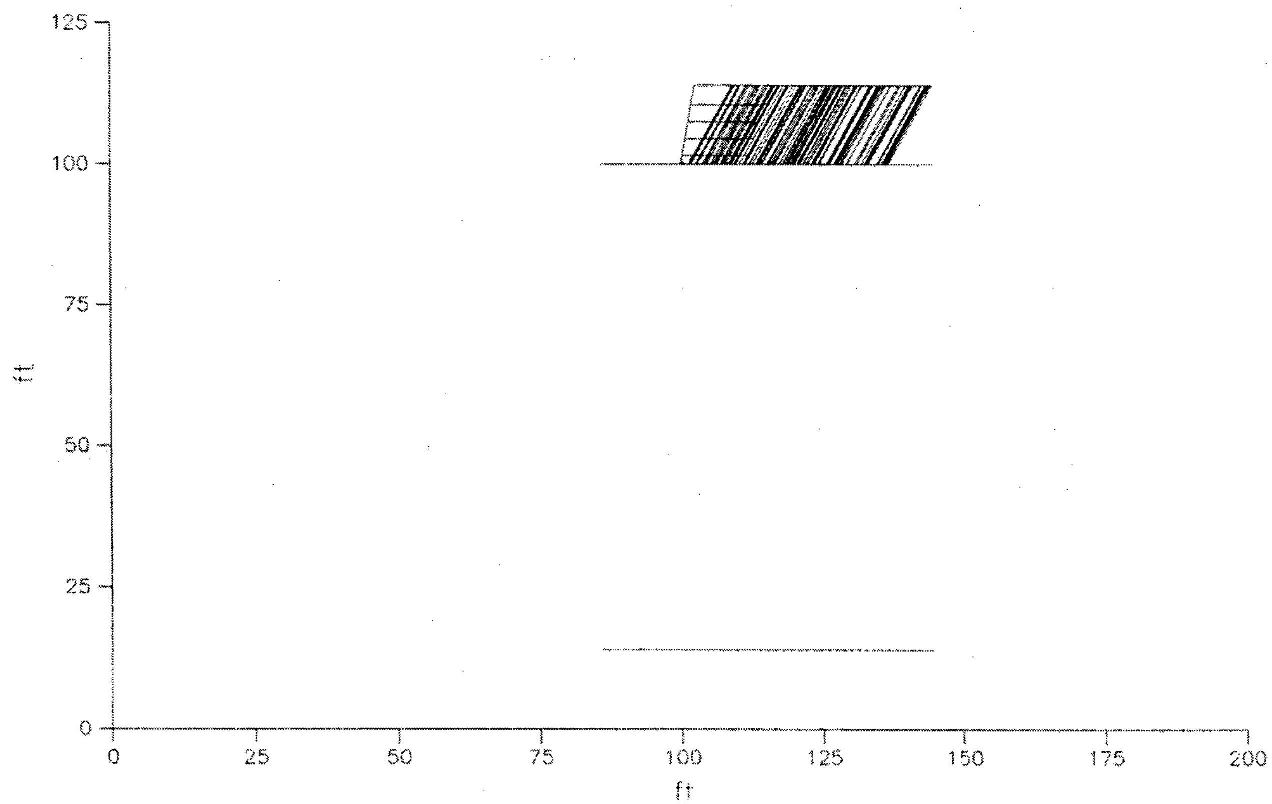
Title : Reinforced Earth Wall
Description : 14' High with Leeward Face approx 80 degrees

Reinforcement Analysis - Most Critical Surfaces



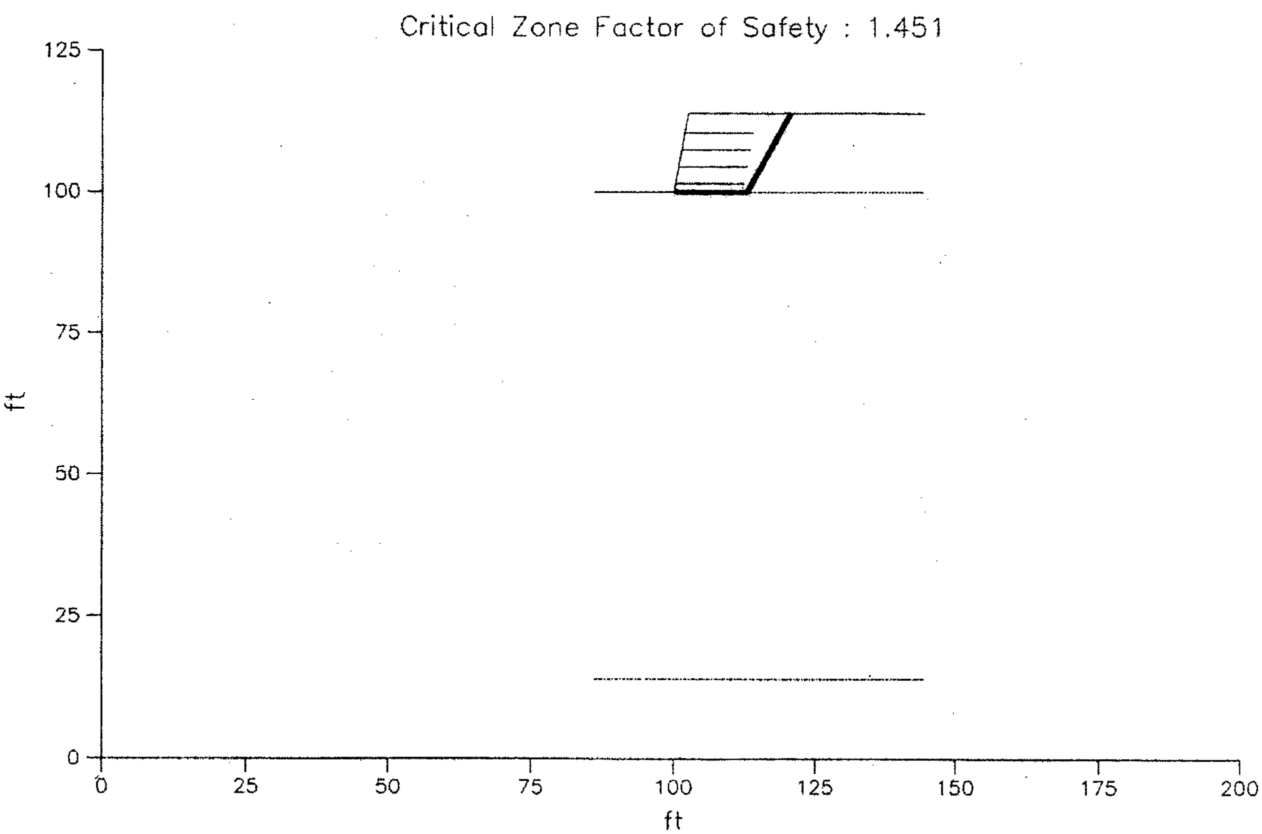
Title : Reinforced Earth Wall
Description : 14' High with Leeward Face approx 80 degrees

Reinforcement Analysis - Search for Critical Zone



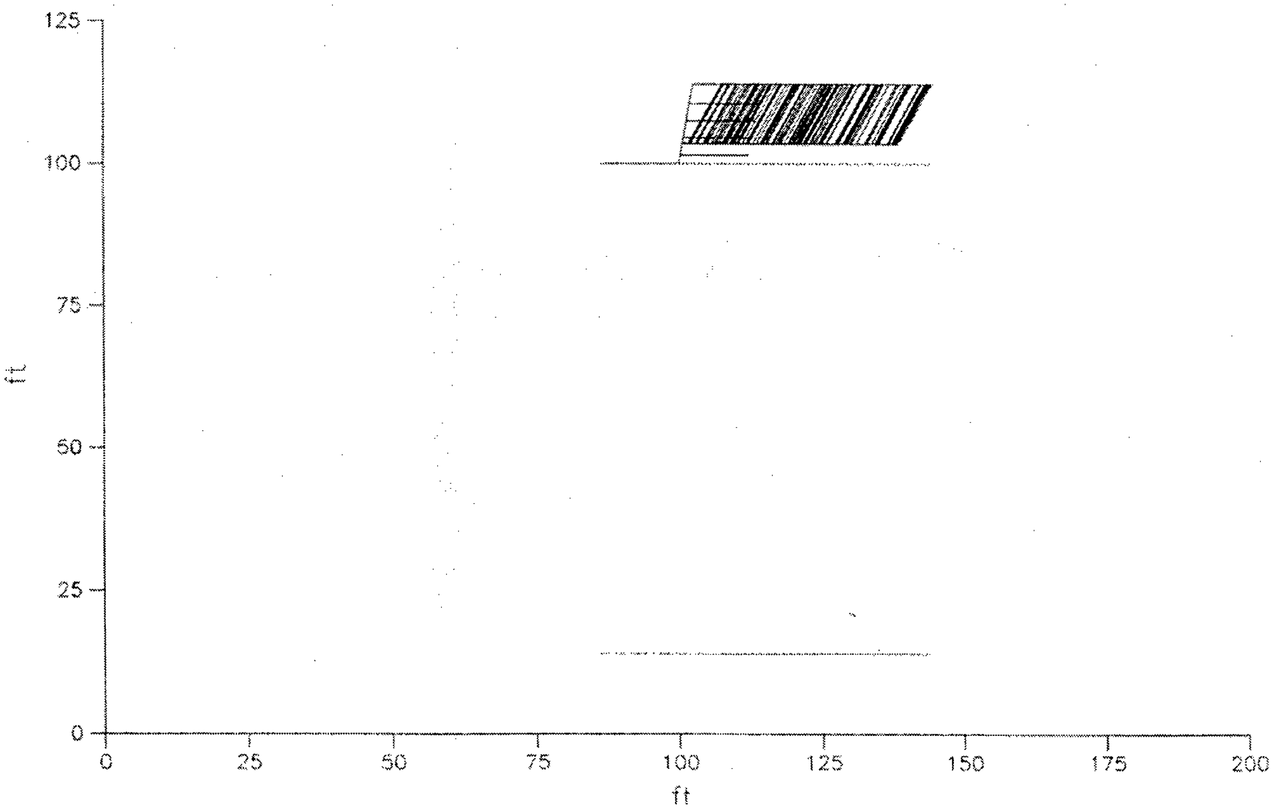
Title : Reinforced Earth Wall
Description : 14' High with Leeward Face approx 80 degrees

Reinforcement Analysis - Critical Zone



Title : Reinforced Earth Wall
Description : 14' High with Leeward Face approx 80 degrees

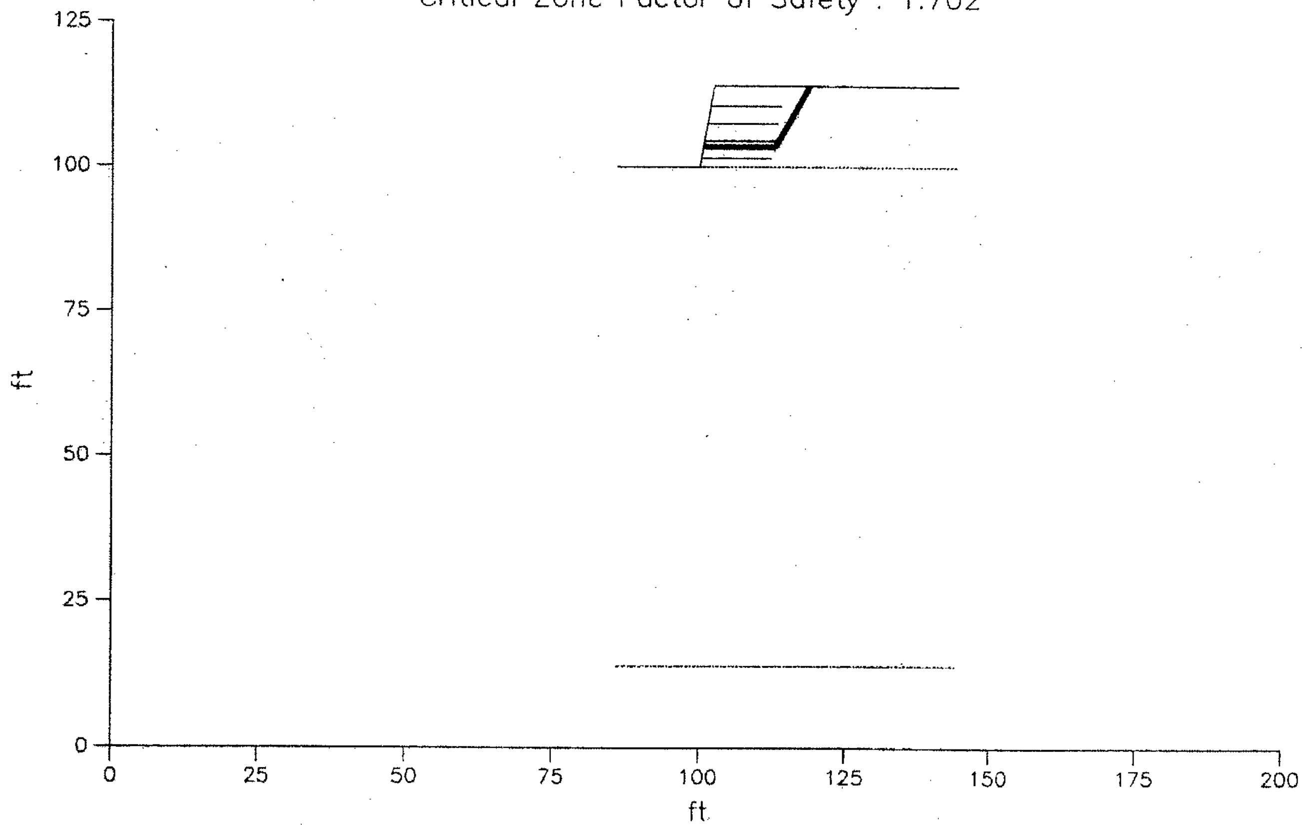
Reinforcement Analysis - Search for Critical Zone



Title : Reinforced Earth Wall
Description : 14' High with Leeward Face approx 80 degrees

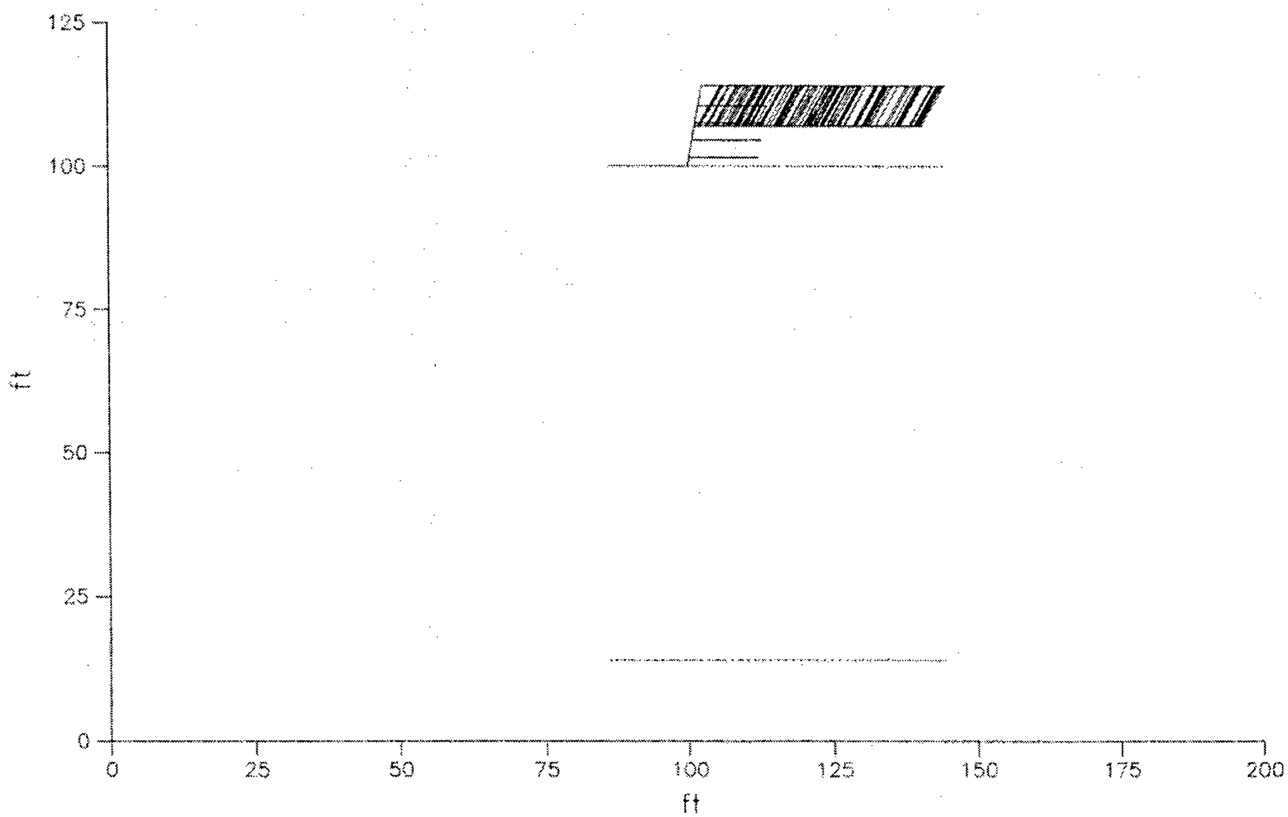
Reinforcement Analysis - Critical Zone

Critical Zone Factor of Safety : 1.702



Title : Reinforced Earth Wall
Description : 14' High with Leeward Face approx 80 degrees

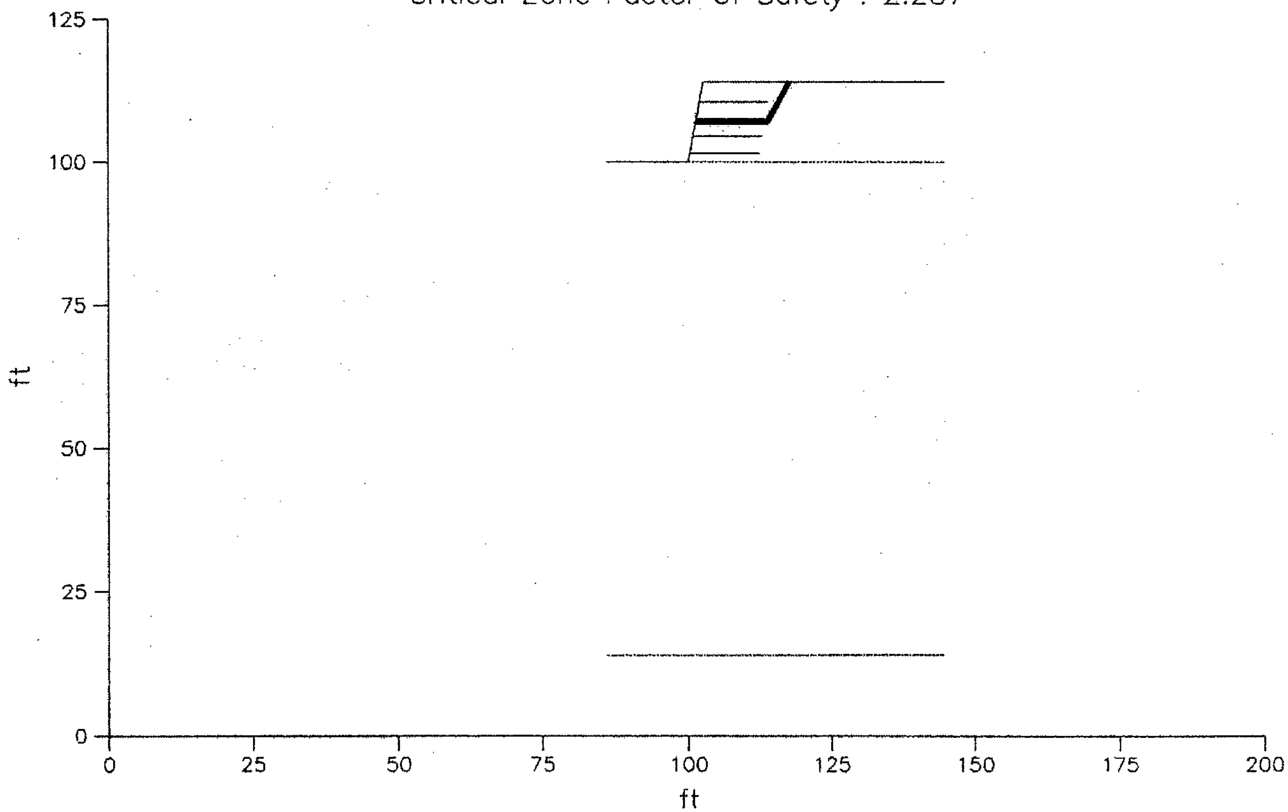
Reinforcement Analysis - Search for Critical Zone



Title : Reinforced Earth Wall
Description : 14' High with Leeward Face approx 80 degrees

Reinforcement Analysis - Critical Zone

Critical Zone Factor of Safety : 2.237



APPENDIX B

TRIAXIAL TEST DATA

Test 1

Chamber Pressure: 14 psi
Area: 6.157522 in^2
Height: 5.53125 in

Time (sec)	Load Cell (volts)	LVDT (volts)	Press.Trans.	Load Cell$_C$ (lb)	LVDT$_C$ (in)	Deviator Stress (psi)
0	-0.00173	-8.81589	-0.00291	-1.73	0	-0.28096
50	0.00071	-8.73344	-0.00307	0.71	0.008245	0.115306
100	0.00361	-7.58608	-0.00435	3.61	0.122981	0.586275
150	0.00488	-7.53952	-0.00642	4.88	0.127637	0.792527
200	0.0174	-7.38807	-0.00607	17.4	0.142782	2.825812
250	0.0645	-7.25694	-0.00488	64.5	0.155895	10.47499
300	0.09894	-7.12978	-0.00456	98.94	0.168611	16.06815
350	0.12199	-7.01127	-0.0047	121.99	0.180462	19.81154
400	0.1401	-6.88688	-0.00428	140.1	0.192901	22.75266
450	0.15601	-6.75791	-0.00494	156.01	0.205798	25.33649
500	0.17057	-6.62478	-0.00479	170.57	0.219111	27.70108
550	0.18419	-6.48987	-0.00475	184.19	0.232602	29.91301
600	0.19833	-6.34934	-0.00473	198.33	0.246655	32.20939
650	0.21189	-6.20476	-0.00462	211.89	0.261113	34.41157
700	0.22518	-6.06086	-0.005	225.18	0.275503	36.56991
750	0.23783	-5.91777	-0.00454	237.83	0.289812	38.62431
800	0.25005	-5.77402	-0.0048	250.05	0.304187	40.60887
850	0.26194	-5.62742	-0.00476	261.94	0.318847	42.53984
900	0.27414	-5.47436	-0.00489	274.14	0.334153	44.52116
950	0.28545	-5.32202	-0.0053	285.45	0.349387	46.35794
1000	0.29604	-5.16923	-0.00538	296.04	0.364666	48.07779
1050	0.30588	-5.01682	-0.00519	305.88	0.379907	49.67583
1100	0.31551	-4.86422	-0.00565	315.51	0.395167	51.23977
1150	0.32426	-4.71215	-0.00556	324.26	0.410374	52.6608
1200	0.33227	-4.5596	-0.00507	332.27	0.425629	53.96165
1250	0.3398	-4.40696	-0.00539	339.8	0.440893	55.18454
1300	0.34966	-4.19595	-0.00488	349.66	0.461994	56.78583
1350	0.37908	-3.09806	-0.00504	379.08	0.571783	61.56373
1399.5	0.38501	-1.95559	-0.00497	385.01	0.68603	62.52678
1432.5	0.37924	-1.19378	-0.00482	379.24	0.762211	61.58971

Test 2

Chamber Pressure: 28 psi
Area: 6.048057 in^2
Height: 5.4375 in

Time (sec)	Load Cell (volts)	LVDT (volts)	Press.Trans.	Load $Cell_C$ (lb)	$LVDT_C$ (in)	Deviator Stress (psi)
0.25	0.00465	-8.93008	-0.00188	4.65	0	0.768842
50	0.00539	-8.88926	-0.00268	5.39	0.004082	0.891195
100	0.00537	-8.83887	-0.0023	5.37	0.009121	0.887888
150	0.00664	-8.78744	-0.00244	6.64	0.014264	1.097873
200	0.00731	-8.74129	-0.00216	7.31	0.018879	1.208653
250	0.00817	-8.68681	-0.00229	8.17	0.024327	1.350847
300	0.00975	-8.62845	-0.00245	9.75	0.030163	1.612088
350	0.01125	-8.56894	-0.00266	11.25	0.036114	1.860102
400	0.01198	-8.51114	-0.00272	11.98	0.041894	1.980802
450	0.01274	-8.44738	-0.00297	12.74	0.04827	2.106462
500	0.013	-8.38293	-0.00284	13	0.054715	2.149451
550	0.01322	-8.31835	-0.00312	13.22	0.061173	2.185826
600	0.01357	-8.23487	-0.00386	13.57	0.069521	2.243696
650	0.02741	-8.14567	-0.00346	27.41	0.078441	4.532034
700	0.08092	-8.05724	-0.00387	80.92	0.087284	13.3795
750	0.11316	-7.97612	-0.00388	113.16	0.095396	18.71014
800	0.1406	-7.88982	-0.00402	140.6	0.104026	23.24714
850	0.15928	-7.80511	-0.00335	159.28	0.112497	26.33573
900	0.185	-7.70339	-0.00342	185	0.122669	30.58834
950	0.2091	-7.60102	-0.00358	209.1	0.132906	34.57309
1000	0.23254	-7.49762	-0.00345	232.54	0.143246	38.44871
1050	0.25563	-7.39429	-0.00365	255.63	0.153579	42.26647
1100	0.299	-7.19472	-0.00355	299	0.173536	49.43737
1150	0.35839	-6.9019	-0.00368	358.39	0.202818	59.25705
1200	0.41503	-6.60932	-0.00381	415.03	0.232076	68.62204
1250	0.46857	-6.31647	-0.00385	468.57	0.261361	77.47447
1300	0.51821	-6.02342	-0.00369	518.21	0.290666	85.68207
1350	0.56364	-5.72945	-0.00371	563.64	0.320063	93.19357
1400	0.60777	-5.41894	-0.00399	607.77	0.351114	100.4901
1450	0.65821	-4.98442	-0.00413	658.21	0.394566	108.83
1500	0.69986	-4.54991	-0.0041	699.86	0.438017	115.7165
1550	0.73161	-4.11603	-0.00442	731.61	0.481405	120.9661
1600	0.75363	-3.67892	-0.00414	753.63	0.525116	124.607
1650	0.76649	-3.05715	-0.00466	766.49	0.587293	126.7333
1700	0.7276	-2.33791	-0.00452	727.6	0.659217	120.3031